# Bond Strength Performance Characteristics of Brick – Mortar Interfaces

**Alexander Miles Seaton**

A thesis submitted in partial fulfilment of the requirements of
Sheffield Hallam University
For the degree of Doctor of Philosophy

March 2004

# Abstract

This thesis embodies the experimental work conducted as part of a research programme undertaken within the Brickwork Laboratories at The School of Environment and Development, Sheffield Hallam University. The work uses a direct tensile test approach, developed in-house, to measure the bond strength of brick-mortar interfaces. The thesis postulates that direct tensile bond strength values can be used as a means of assessing compatibility of brick and mortar properties which directly influence the structural and durability performance of brickwork. The work identifies that direct forms of tensile testing are suited for comparative research more readily than flexural testing techniques, which induce inherent variability to the test system due to sample format and by application of the associated flexural bending theory. It is maintained that flexural bending tests reflect parameters which influence disproportionately the compressive strength of the mortar and the joint periphery. The work uses traditional volume ratios for mortars, proportioned to a constant mass of sand, to identify the effect that discrete changes in cement and lime content have upon bond strength performance of the mortar. The results show that cement content of mortar has no significant influence on bond strength, provided that the combined proportion of cement and lime maintain a 1:3 ratio by volume with the sand. Furthermore, it has been shown that the volume of the mix water should match the volume of cementitious material, in order to achieve suitable workability. The work produces a bond strength development curve for samples aged between 5-minutes and 2-years of age and concludes that bond strength does not develop in the same manner to compressive strength and that bond strength may decline post 28-days. Samples up to 2-years in age can demonstrate up to 40-percent loss of bond due to the effects of sustained drying shrinkage. Consequently the work questions the value of using 28-day strength tests as a means of predicting future bond strength performance. It is identified that the controlling parameter which effects bond strength development is the removal of the excess mix water from the mortar by brick suction forces. The work examines unit water absorption characteristics and identifies that the initial rate of suction test is not sufficiently representative of a unit's ability to remove water from a retentive mortar bed. In response a unique method, which measures the continuous water uptake of the brick bed-face is presented. The resulting water absorption profile identifies the rate of change of flow and the resulting force function, with which water is potentially extracted from the retentive mortar bed. Results show that a good correlation between a unit's suction force and bond strength can be attained. It is presented that initial bond strength is developed by volumetric plastic shrinkage of the mortar bed, induced by rapid removal of the excess mix water by brick background suction, which generates a mechanical lateral gripping action to the undulations of the brick bed-face.

For Louis

I wish to thank my Father Ray Seaton for his relentless support and dedication during the turbulent years that this thesis has been in the making

and in memory of my late Mother Shirley Seaton I wish to dedicate these works

and to my Brother Mark Seaton for his faith and encouragement throughout

and to my good friends and colleagues along the way, Eshref, Finbar, Sophie and Mathew

Thank you

I hereby declare that no portion of the work referred to in this thesis has been submitted in support of an application for another degree or qualification of this or any other university or other institute of learning. All sources of information have been duly acknowledged.

**A.M. Seaton**

**March 2004**

# Acknowledgement

I wish to acknowledge the pioneering work of Mr Alan Taylor-Firth and thank him for his early input into this research project.

I would like to thank also Dr Elizabeth Laycock for her help and encouragement in the preparation of this thesis and also Dr Jamal Khatib.

I wish to make a special thank you to Mr Steve Hetherington for his dedication and technical assistance and also to the late Mr Graham Coe.

Thanks are also given to Mr Barry Pratt for information and advice.

# Contents

# 1.0 Introduction

The evolution of the modern masonry wall finds its origins in the most early forms of human habitation, with the fundamental nature of its construction evolving little until the latter part of the $19^{th}$ Century. Historically, the essential feature of masonry was one of robust, load-bearing stone or clay units, set usually in lime-sand mortar and constructed from indigenous materials by local craftsmen, intended to suit the purpose of location and nature of dwelling occupation.

The perceived function of the mortar was essentially to keep the units apart, fulfilling a passive role of distributing compressive stress between units of non-uniform shape and size. The mortar gained strength slowly through a process of carbonation. The accepted weakness of the tensile strength of masonry was compensated for by substantial wall thickness and utilisation of the materials self weight. Furthermore, it was generally accepted that such construction was not weatherproof and wall thickness alone was relied upon in order to prevent rain and wind penetration.

The $20^{th}$ Century has placed considerable demands upon the structural form and durability performance requirements of the building envelope. The ethos for pursuit of low cost, but increasingly architecturally challenging designs, coupled with a change in user requirements, has brought a fundamental transformation to the way in which masonry is used in construction.

Masonry in the UK today typically takes the form of non-load bearing cavity external walls of 100mm thick brickwork, a cavity of between 50mm and 100mm and a load-bearing blockwork inner leaf. Both leaves are of uniform course and thickness, built in stretcher bond using cement and sand mortar. The additional hardware required to achieve such a form of construction will include stainless-steel cavity ties, metal restraint straps, steel wind posts and cavity trays over openings. Invariably the inner and outer leaf of the cavity wall will be made from materials of different mechanical properties, resulting in differential thermal and moisture movement; this is further exacerbated by varying levels of thermal insulation incorporated within the cavity. In consequence there is a requirement for frequent horizontal and vertical movement joints with sliding stainless steel anchor ties to allow the outer leaf to move and chemically advanced sealants to prevent water ingress. Sutherland[1] maintains that the masonry industry has accepted each apparently beneficial change as itself immutable and gone on reacting positively to the problems caused by it, thus building up a string of sequential benefits, problems and solutions. He argues that the modern cavity wall is a complexity which needs to be compared to the simple bonded construction of earlier years.

In order to consider the reasons for such a dramatic change in the form of masonry construction and materials over a relatively short period of time in respect to its long and established history, it is beneficial to examine the genesis of the masonry wall and to consider the demands which have been placed upon the fabric and structure of the building, both by the physical and socio-economic environment.

## 1.1 Evolution of The Cavity Wall

The principal disadvantage with traditional form of masonry construction was that buildings were often cold and damp. As populations increased, wall construction, which could be anything over one and a half feet thick (0.45 m), amounted to considerable "wasted" space. According to Sutherland[1], walls could occupy 10-15% of the overall plan area of the building.

During the 19th Century increasing emphasis was placed on low cost housing for the working classes. Piers[2], reporting on wall construction in Belgium, suggests that around 1850, Belgium dwellings had three classes of wall thickness, paralleled closely by the social class of the occupants; the working mans house had walls of one brick thick (20cm), the middle class house one and a half brick thick (30cm) and the mansions of the upper class had two brick thick walls (40cm). While the two brick thick wall could be considered weatherproof, the one and a half thick wall would be damp probably once every 5 years and the single brick wall remained permanently saturated. A similar comparison can be drawn with the condition of the housing stock in Britain in 19th and early 20th Century. Consequently, the health of the nation and in particular the health of the workforce, was deemed to be directly related to the quality of the housing stock. The Victorians adopted the rudiments of the modern cavity wall as a panacea to cure dampness.

Various forms of walling were constructed to introduce an air cavity into the wall, but invariably this cavity was breached in one way or another by the need to maintain structural stability. Connection of two single leaves by metal ties was known as early as 1860, but cavity wall construction in this manner did not catch on until the period between the two World Wars. This was prompted by The Tudor Walters Report of 1918[3] which saw the cavity wall as part of the universal prescription for healthy and comfortable living.

## 1.2 Environmental Performance Requirements

The socio-economic climate over the past century has dictated not only the structural form of construction in which masonry is utilised, but has also proliferated demands for improved thermal performance of the wall material. During the last half of The 20th Century, UK governments have progressively introduced legislation into the Building Regulations in an attempt to improve the thermal performance of buildings. Early legislation was driven by a need to improve the durability performance of the building fabric and to control condensation and mould growth. Later legislation has reflected an increasing awareness of the adverse environmental impact of energy consumption and $CO_2$ emissions. In particular it was identified that domestic heating and hot water accounts for 25 percent of all $CO_2$ emissions produced in the UK and consequently, buildings have been targeted as one of the principal contributors to "greenhouse gases".

Following the outcome of The Earth Summit in Rio de Janeiro in 1992, all developed countries were set targets for reducing emissions of greenhouse gases to 1990 levels by 2000. Further legally binding cuts in greenhouse gas emissions were introduced in 1995 at The Kyoto Earth Summit, which aimed to cut emissions by an international average of 5.2 percent from 1990 levels by the year 2010.

### 1.2.1 Thermal Insulation requirements

The mandatory thermal insulation requirements for walls was first introduced into the Building Regulations in 1966. At the time the requirement was for a U-value of 1.7W/m² °C, which could readily be achieved by conventional two leaf brick cavity wall, finished internally with normal plaster. The reduction of the U-value to 1.0 W/m² °C in 1976 accelerated the introduction of lightweight concrete masonry products for the inner leaf, in order to provide the necessary insulation. Beard[4] maintains that a further reduction of U-value to 0.6 for housing and introduction of 0.6 and 0.7 for some non housing applications in 1982, effectively eliminated any viable cavity construction reliant upon two skins of masonry alone and thereby accentuated the introduction of lightweight cavity insulation into the cavity space.

Further amendments to Building Regulations[5] for England and Wales were introduced in July 1995 and outlined a basic U-value requirement for exposed walls of 0.45W/m² °C. Current compliance with Part L of The Building Regulations for England And Wales[6] requires a U-value for exposed walls of 0.35W/m² °C.

A further consequence of the introduction of insulation into the cavity space has resulted in the outer leaf of the cavity becoming exposed to more extremes of temperature; the drainage path of the external leaf has been restricted predominantly to the outer face of the wall. Walls with high levels of cavity insulation are more likely to experience interstitial condensation and exaggerated thermal and moisture movement, together with increased risk of frost action and degradation of the mortar through sulphate attack. Of most relevance, the inclusion of insulation and other hardware material within the cavity provides a means of transmitting moisture through to the internal leaf. As a result, durability problems manifested by water penetration, salt staining, frost spalling, sulphate attack and movement cracking remain the principal material defect in masonry structures.

### 1.2.2 Rain Penetration

Cavity walls require very high standards of workmanship in order to avoid defects such as incorrectly installed wall ties or ties sloping inwards, mortar droppings on ties, debris bridging the cavity, incorrectly installed cavity trays or missing drain holes. Cavity insulation, whether filling the cavity in its entirety or leaving a nominal cavity between the insulation and the outer leaf, merely serves to exacerbate an already common defect. Whether in isolation or accumulation, these defects lead to rain penetration. The problem was highlighted in a NHBC newsletter[7] in 1984 which stated, "do not use cavity wall insulation unless your building standards are very high".

The National House Building Council (NHBC)[8] reported £4.5 million in claims relating to rain penetration as a result of the severe storms of December 1989 and January and February 1991. Although these storms were severe, meteorological data showed that such intensity had been exceeded previously on a number of occasions. Of even more concern was that the worst affected areas were the counties of Hampshire, Kent and Wiltshire, which were not recognised as particularly exposed areas according to Driving Rain Index maps[9].

The NHBC inspectors found that cavity walls which were fully filled with insulation were responsible for three-quarters of the above claims; however 1 in 7 of the claims related to "clear" cavities, or cavities in which insulation material did not occupy the full thickness. The NHBC reported significant damp penetration problems in 1023 dwellings, 80% of which were in buildings less than 12-months old, and determined that defects which gave rise to water crossing the cavity were mortar droppings on wall ties, ties slopping towards the inner leaf, droppings at the base of the wall bridging the dpc, trays and dpc's incorrectly formed and perp-end joints which were not fully filled.

In addition, research[10] has shown that retro-fill insulation can potentially lose its water repelling properties over time, since rain water penetrating the facing brickwork could become alkaline, breaking down the waterproofing properties.

Newman and Whiteside[11] investigated the penetration of water through 100mm single leaf brickwork, such as that found in the external leaf of a cavity wall. It was shown that after wetting at a rate corresponding to severe storm, a high proportion of water was able to pass through the wall which from visual inspection of their external face, could be said to be "well built". It was confirmed that leakage occurs primarily through cracks between brick and mortar at their interface. Measurement of air leakage indicated that these cracks were on average approximately 0.1mm wide. The results showed that no leakage occurred through the mortar itself, 7% through the bricks and 40% and 54% through the interface cracks at the bed-face and perp-end respectively.

Newman and Whiteside argue that greater part of cracking at brick-mortar interface is caused by suction of water from the wet mortar after laying; resulting in mortar shrinkage. In addition bricks and mortar undergo small reversible dimension changes with changes in moisture content. This may contribute to the widening of the cracks as the wall dries out and narrowing as the wall becomes wet. Leakage is sensitive to such changes, explaining why incidence of rain penetration is much more common in new dwellings, with reducing incidence as the building matures.

Researchers, for example Skeen[12], Butterworth and Skeen[13] and Grimm[14] appear to be in broad agreement that the main mechanism for rain leakage through masonry walls is by flow of water through micro-cracks at the interface between the units and the mortar, while capillary flow through porous units is a secondary mechanism.

## 1.3 Structural Performance Requirements

Conventionally the suitability of materials for their intended purpose have been evaluated and specified in terms of their individual compressive strengths; the structural design of such load-bearing elements being based upon prescribed wall thickness, given by a denomination of standard number of units.

A consequence of the introduction of the cavity wall is that brickwork today predominantly takes the form of non load-bearing single skin, 100mm thick brickwork, spanning horizontally between floor and roof restraints, or, as in the case of brickwork cladding to framed buildings, as laterally spanning wall panels. Resultantly, walls are much more slender and flexural and tensile forces, which in the past were not explicitly designed for, must now form part of the design criteria.

Tensile forces in the wall can be developed in two ways. Flexural tensile forces may be caused by lateral wind or earth pressures or as a result of eccentric load application onto the wall. Direct tensile forces may be generated by differential settlement of part of the building or uplift forces due to wind suction acting on the roof structure.

The flexural tensile strength of masonry may be defined as the ultimate tensile stress developed in the extreme edge of the bed-joint when subjected to bending about an axis parallel to the bed-joint. The self weight of the wall above the critical bed-joint, together with building loads supported by the wall, will compensate the vertical tensile stress developed within the bed joint. Horizontal cracking along the bed-joint and bulging of the wall are indicative of failure induced by flexural bending.

Flexural tension can be developed in horizontally spanning masonry, due to an application of lateral loading inducing bending perpendicular to the bed-joint. Such application of load induces flexural tension in the perp-end joints and torsional shear forces within the bed-joint.

Shear stresses may also be developed within the bed-joint, due to thermal and moisture movements within the wall, stress concentrations around openings, the long-term effects of creep, settlement or lateral movement of the building frame inducing racking shear.

Grimm[15] argues that shear strength of brickwork is a function of the bond strength of the mortar to the unit and the frictional resistance at the brick-mortar interface.

A further structural consideration is the effect that compressive loading has on the structure. Grimm[15] explains that the uni-axial compressive strength and modulus of elasticity of mortars are usually considerably lower than the corresponding values of brick, while Poisson's ratio is higher. Accordingly, under axial compression the unrestrained lateral strain of mortar would exceed that of brick. However, shear and friction between brick and mortar restrain the mortar, placing it in tri-axial compression, while the brick is in axial compression and lateral tension. Thus the typical uni-axial compressive strength of the brick masonry exceeds the uni-axial compressive strength of the mortar. Full-scale tests on walls show that compressive strength of brickwork is typically 30-40 % the crushing strength of the individual units, demonstrating that the compressive strength of the mortar has little influence on the strength of the brickwork.

## 1.4 Structural Design and Code Requirements

The British Standard Code of Practice for Structural Use of Un-reinforced Masonry, BS 5628[16], specifies the structural performance properties of masonry in terms of characteristic compressive strength $f_k$, characteristic flexural strength $f_{kx}$ and characteristic shear strength $f_v$. The code gives both characteristic compressive and flexural strength values for categories of unit and mortar based on empirical data from large scale laboratory test programmes on panels of representative components tested at 28-days post manufacture.

The code of practice makes no reference to the determination of direct tensile bond strength. The only guidance is given in clause 24.1 of BS 5628[16] which states "the characteristic flexural strength, $f_{kx}$, should be used only in the design of masonry in bending. In general, no direct tension should be allowed in masonry. However, at the designer's discretion half the flexural strength values may be allowed in direct tension when suction forces arising from wind loads on roof structures are transmitted to masonry walls, or when the probable effects of misuse or accidental damage are being considered".

The fact that the structural code for masonry does not permit the development of any direct tensile forces clearly infers that masonry structures are still to be perceived as compression structures, with brickwork forming, for all intent and purpose, a vertical pile of units.

Clearly, this cannot be the case given the increased slenderness of masonry and the structural and durability requirements now placed upon the building envelope. It is apparent that "adhesion" between the brick and the mortar does serve a structural purpose, but due to the difficulty in ascertaining quantifiable data, the tensile nature of masonry remains conjectural. Perhaps one reason for lack of design information is the inherent variability encountered in all forms of bond testing and the absence of a universally accepted tensile testing methodology. This discussion will be expanded further in Chapter 2.

There is a general acceptance amongst masonry designers that masonry does not support tensile forces. In turn, direct tensile forces rarely occur in masonry or at least can be readily designed out or incorporated by using large partial material safety factors (typically 2.5–3.5) and accepting low design strengths. In consequence, characteristic tensile bond strength of masonry has little relevance to the designer.

This absence of informative design and specification information reflects the lack of understanding of the bonding mechanism. More importantly, it undervalues the contribution that tensile bond potentially plays to the durability performance of brickwork as a composite material.

The structural and durability performance requirements demanded from masonry in the last fifty years have dictated a fundamental change to the nature of interaction between the unit and the mortar. It is presented that the function of mortar as a component within the modern cavity wall, is required to bond the units together, to present a watertight composite material that is capable of transferring induced tensile stresses. Failure of mortar to fulfil these functions will lead to water penetration, with subsequent impact on durability and loss of structural integrity. This induced performance requirement of mortar is far removed from the traditional perceived function of mortar, which was discussed at the beginning of this chapter.

## 1.5 Research Objective

In response to the perceived interaction requirements between the brick and mortar, the work embodied in this thesis provides the opportunity to evaluate bond strength performance characteristics of brick-mortar interfaces. The pursuit of the hypothesis is intended to contribute significantly to the present knowledge of the bond formation mechanism.

It follows that the quality and nature of the contact at the interface between the brick and the mortar, characterised by the magnitude of the measured bond strength, is indicative of the compatibility of the component materials. Compatibility between bricks and mortar not only affects the future performance of the composite but also promotes construction productivity and beneficial bricklaying practices.

Material compatibility is increasingly relevant in the masonry industry, reflecting the developments in construction philosophy. For example, the design of mortar mixes have become increasingly sophisticated and in 1989 it was estimated[17] that there were 137 different mixes available for specification. Superimposed upon the increasing range of mortar constituents, additives and unit type, the potential combinations of materials become limitless.

The methods for evaluating masonry performance have not kept pace with the technological advancements in the material market and construction practices. There is an increasing need for more informed understanding of the bond strength development mechanism and a satisfactory means of testing and evaluation of bond strength performance.

The pursuit of an understanding of bond development process has produced a considerable volume of literature on the subject. It is the authors opinion that conflict in the literature exists regarding the mechanism of the bond formation process and the parameters which influence its development. Examples of such disparity in the research will be discussed in the literature reviews contained within each chapter of this thesis.

It is recognised that there is no universally accepted tensile testing methodology for masonry and research has tended to vacillate between testing small representative masonry wall panels such as wallettes, or couplets or prisms, by either the application of flexural bending or by the application of a direct tensile force.

It is the nature of this applied force to the masonry assemblage during bond testing, together with the subsequent application of the associated stress theory used for the derivation of the bond stress, which promotes much of the uncertainty surrounding parameters contributing to bond strength performance. The author maintains that while flexural bending tests provide good simulation of in-situ tensile stresses likely to be encountered by masonry under lateral loading, they are not representative of the true tensile bond strength between the brick and mortar interface. Mortar is an anisotropic material; displaying different mechanical properties in tension and compression. The application of flexural bending depends upon the mortar in compression zone forming a hinge to enable the mobilisation of tensile stresses. While the interaction between the compressive and tensile behaviour is important to the structural performance of masonry, the existence of these dual parameters does not allow for the identification of bond strength parameters in isolation.

Chapter 2 appraises the chosen tensile testing methodology and draws comparisons between flexural methods of testing and those which rely upon the application of a direct tensile force and concludes that comparative work is more sensitive to direct forms of tensile testing than the less searching nature of flexural testing.

The application of flexural bending tests, for reasons mentioned above, has resulted in much of the research drawing comparison between the development of the compressive strength of mortar with that of bond strength. This has attributed bond strength development to an association with the cement content of mortar and has established the criteria for testing bond strength performance characteristics at 28-days, in accordance with compressive strength gain of mortar.

Chapter 3 investigates direct tensile bond strength performance of generic mortar mix proportions. By proportioning cement and lime to a constant mass of sand, using traditional volume ratios, it is possible to investigate the influence of increasing cement content of mortars upon bond strength. Correspondingly, the addition of lime can be monitored independently as an additive. The work identifies a 1:1:6 optimum mix when reviewing bond strength performance and challenges previous work which draws comparison between bond strength and compressive strength development of the mortar.

The 28-day benchmark for testing masonry in tension may prove to be theoretical if it can be shown that bond develops independently to the compressive strength of the mortar. Such a finding would hold significant implications as to the validity of research already undertaken on bond strength tested at 28-days. Moreover, the age of testing becomes an important criteria if bond strength is to be promoted as a performance predictor of component compatibility.

Chapters 4 and 5 address the bond strength development with time and distinguish two distinct bonding phases for mortar in the wet state between 5-minutes and 24-hours post manufacture and mortar in the hardened state between 2-days and 2-years of age. The experimental work identifies that the bond mechanism is marked by rapid plastic shrinkage of the mortar bed in the early phase, which can be advantageous to the bond formation. This initial stage is then followed by prolonged drying shrinkage in the latter stages, which, if excessive, may lead to the impairment of bond over time.

Chapter 6 recognises that the removal of excess mix water, by brick background suction contributes to both early plastic shrinkage and subsequent long-term drying shrinkage. An experimental programme compares conventional methods of assessing unit suction forces and methods of adjusting unit suction rates and reveals that they are not sufficiently sensitive enough to predict both the quantity and the way in which water is removed during the crucial bond formation stage.

Chapter 7 develops a unique method of measuring the continuous water absorption of a unit and identifies suction force, as opposed to quantity of water removal, as the critical parameter which determines the extent of excess mix water abstracted from the retentive mortar bed.

Chapter 8 concludes that bond strength development between the brick and the mortar is indicative of component compatibility. Quantification of bond strength, measured by a direct tensile testing approach, helps to identify those parameters which influence the bond formation. Having identified these parameters, an explanation is presented as to the actual mechanism of bond formation.

## 1.6 Qualifications

There exists inevitable variation in masonry bond strength testing, induced by the chosen testing methodology, sample manufacture and curing. Overlaid upon this is inherent variability in mortar constituents, mortar batching and unit characteristics. In consequence, experimental design which seeks to identify contributing parameters must produce high sample representation in order to promote statistical analysis.

In response, this study has purposefully controlled sample variation by applying consistent mortar batching and mixing procedures which utilise a constant mass and source of sand, together with prescribed mix water content. Mortar types investigated have been limited to the generic mortar mixes which appear in Table 1 of BS5628 Part 1[16]. Similarly the experimental work has used one type of clay masonry unit for the evaluation of bond strength performance. Couplet samples used for testing represent only bed-face bonding characteristics and the performance of the perp-end bond have not been investigated. Testing of isolated couplet joints do not take into account the influence of pre-compression due to overlying courses. Workmanship parameters have been controlled by the use of a couplet joint forming jig and samples throughout the entire test programme have been produced by the same operator.

It is the authors view that the effectiveness of this style of research programme, which seeks to promote an understanding of the primary bond formation mechanism, must restrict the number of different unit types and mortar mix combinations to those that are readily available. There is considerable temptation to include units with different characteristics such as calcium silicate or concrete units into a study, or to include masonry cement mortars or mortars with additives. This will only serve to detract from the pursuit of an understanding of the bond formation processes at work. The methodology, once established, may then be applied to further investigations which serve to incorporate more contemporary technologies.

## References


1. Sutherland, R.J.M. Have We Got It All Wrong. British Masonry Society Proceedings, No. 5, 1993.
2. Piers I.G. Continental Cavity Walls Are Sometimes Different. Proceedings of British Masonry Society, No. 5, 1993
3. Local Government Board. Report of the Committee Appointed to Consider Questions of Building Construction in Connection with the Provision of Dwellings for the Working Classes (Tudor Walters Report). Cmnd.9191, HMSO, 1918.
4. Beard R. Three Performance Requirements Of A Cavity Wall. J. Brit. Ceram. Soc., 83, (6), 1984.
5. Department of the Environment. The Building Regulations. (Amendment) 1994. London, HMSO, SI1994/1850.
6. Department of the Environment. The Building Regulations: Part L. April 2002.
7. National House Building Council. Newsletter No. 4. 1984.
8. National House Building Council. Standards Extra. September 1991.
9. British Standards Institution. Code of Practice for Assessing Exposure of Walls to Wind Driven Rain. BS8104:1992.
10. Kirtschig,K. Wolf-Rudiger. Investigation of Long Term Behaviour Of Masonry With Filled Cavities. Proceedings of the 7th International Brick Masonry Conference, Melbourne, Australia, Feb. 1985.
11. Newman, A.J. Whiteside, D. Water and Air Penetration Through Brick Walls-A Theoretical and Experimental Study. Trans. J. Br. Ceramic Soc.,80, 17-26, 1981.
12. Skeen, J.W. Experiments on the Rain Penetration of Brickwork: The Effect of Mortar Type. Trans. Brit. Ceram. Soc., 70, (1), 1971.
13. Butterworth, B. and Skeen, J.W. Experiments on the Rain Penetration of Brickwork. Trans. Brit. Ceram.soc., 61, (9), 1962.
14. Grimm, C.T. Water Permeance of Masonry Walls: A review of the Literature. ASTM Spec. Tech. Pub.778, 1987.
15. Grimm, C.T. Strength and Related Properties of Brick Masonry. Journal of the Structural Division, p217-232, 1975 Jan.
16. British Standards Institution. Code of Practice for the Use of Masonry: Part 1: Structural use of Unreinforced Masonry. BS5268: Part 1:1992
17. Harrison, W.H. and Bowler, G.K. Aspects of Mortar Durability. British Ceram. T.& J., 89, (3), 1990.

# APPRAISAL OF TENSILE TESTING METHODOLOGY

## Chapter Summary

*Chapter 2 describes the principles associated with testing the direct tensile bond strength of brick-mortar interfaces. A review of the associated literature examines the present test methods used to test bond strength and identifies the inherent problems encountered with all forms of masonry bond testing. The reasons for adopting a direct tensile testing approach are explained and the method of determining direct tensile bond is described in detail.*

## 2.0 Introduction

The inherent variability present in masonry units, mortars, workmanship, sample preparation and the testing procedure itself, generate inevitable variance in quantitative bond strength results. High variability has disguised the influence of particular treatments from those induced through uncontrollable factors and therefore has further hindered the understanding of the bond formation process. It is imperative therefore that statistical analysis can be supported by a testing method which will yield a relatively large sample population and which will promote consistency.

The suitability of the testing practice for comparative research depends equally upon the format of the test specimen and the test mechanism. Flexural forms of testing generally rely on determining the strength of the weakest joint in a wallette or stack bonded pier. In consequence, the scale of the specimen format restricts the number of replicates which can practically be manufactured, stored and tested during the investigation of any one particular treatment. Limited sample population, in association with variability induced during the manufacturing process, restricts statistical confidence when assessing the significance of a particular outcome.

In an attempt to generate higher sample representation, a wrench type device has been employed to provide a mean strength of all joints in a prism or wallette, as an alternative to single joint failure obtained during beam bending. However, the influence of mortar spot-board age, variation in workmanship or increasing pre-compression of consecutive courses during the process of sample preparation, all introduce inherent variation into the sample population. Many of the statistical techniques applied to flexural test results from such specimen formats assume normal distribution of sample populations and yet the influence of the processes listed above may form skewed data, with results containing inherent bias.

Another issue concerned with flexural testing of tensile bond strength is the assumption that the neutral axis lies at half the joint depth. Application of beam bending theory will yield an ultimate tensile stress at the periphery of the joint. Imbalance between the tensile and compressive properties of the mortar may result in an eccentric neutral axis, resulting in an over evaluation of the true bond strength. For example, factors such as brick background suction, water-cement ratio and cement content of the mortar will contribute to a modified compression zone, which will have an apparent influence on the numerical evaluation of the tensile bond strength.

Examining the issues outlined above and those contained within the following literature review, this chapter intends to demonstrate that direct forms of tensile testing are more suited to the investigative nature of comparative studies than flexural testing approaches. It is proposed that factors influencing the bond formation process can only be identified using quantification of bond strengths gained from a direct tensile testing approach.

## 2.1 Discussion of Literature Relating to Tensile Bond Testing

The literature reviewed in this chapter is concerned only with methods employed to test and quantify bond strength. The influence that a particular test method may have upon the determination of factors leading to bond strength development is discussed, in order to demonstrate that a chosen test method may have a misrepresentative effect on the contribution made by a particular treatment.

### 2.1.1 Methods of Testing Tensile Bond Strength

In general there are two approaches to testing the bond of masonry units to mortar; those which rely upon the application of an axial tensile force to pull the bond apart using samples made from couplets and those involving the application of a bending moment to the joint by either wallette or beam bending or by applying moment via a wrench type device.

At present, there is no specified test method for testing bond strength on site in any European country. In the UK the only standardised test is the flexural strength wallette test, specified in Appendix A3 of BS 5628[16] and also in ISO/DIS 9652-4[18] and CEN pr EN1052-2[19], which realistically may only be applied to laboratory based studies due to specimen format and sample size.

The flexural strength of un-reinforced masonry is represented by two parameters, the flexural strength parallel to the bed joint and the flexural strength perpendicular to the bed joint. This review is concerned with the measurement of brick bed-face and mortar interface bond and therefore consideration is only given to testing properties parallel to the bed joint.

The Australian stack prism[20], the US stack prism[21] and the bond wrench[22] are further methods of testing in flexural tension. In the USA, the use of the bond wrench in the laboratory is now covered by ASTM Standard C1072[23] and the method is also specified in the Australian Code of Practice AS3700[20]. The Building Research Establishment promote the use of the bond wrench[22] for testing quality control on site, existing masonry, mortar variations, laboratory investigations and bond between mortar and damp proof membrane materials.

The ASTM crossed-couplet test[24] used in the USA and the Sheffield Hallam University test[25] are two methods of testing in direct tension.

The disadvantage for flexural testing of wallettes is that specimen manufacture is costly and time consuming and remains unable to compensate for the variability in workmanship. The effect of pre-compression of varying amounts from courses above and the changing age of the mortar during manufacture are also simultaneous influences. Subsequent transportation and storage of samples may also induce damage to the test specimens.

The Bond wrench test serves to confront many of the problems associated with wallette testing since it may be used on several different specimen formats, including site or laboratory constructed wallettes or couplets. In a paper by De Vekey, Page and

Hedstrom in 1993[26] it was demonstrated that bond wrench measurements have high variability which may be responsible for their lack of acceptance as international standard tests. This paper compared the variability in bond wrench measurements between the UK, USA and Australia and analysis of variance identified the running mean of coefficients of variation (COV's) between 56.5%, 18.05% and 19.6% respectively, for site prepared, laboratory tested specimens from a wide variety of situations.

### 2.1.2 Specimen Format

De Vekey et al[26] reported that in some cases the variability indicated by the standard deviation is relatively constant and in fact it is the mean bond strength that has more influence upon the coefficient of variation. They conclude therefore that strongly bonded masonry gives a lower apparent variability overall, than poorly bonded masonry. The authors also express concern that the results of the bond wrench measurement are sensitive to the position of the joint in multiple joint specimens. They argue that the lower mass of units during laying and curing causes the upper joints to be weaker than the lower joints. In addition they found evidence during quality assurance testing done in the USA that the overall CoV's fell from around 20% to 16% over the course of construction, which can possibly be accounted for by an improvement in workmanship practice. Although this is to be expected on site specimens, it indicates that problems may arise concerning workmanship in any form of sample preparation.

Anderson[27] demonstrated that the strength obtained by testing individual joints in a wallette specimen was greater than that obtained from a single joint failure since joint strength is the mean of a complete sample of joints whereas the wallette strength is the mean of a sample of essentially weaker joints. Anderson estimated wallette strengths as 60% of joint strengths. Lovegrove and De Vekey[28] found that the number of joints in the zone of constant bending moment could influence results. The greater the joint number the lower the bond strength.

Lovegrove[29] determined that the presence of perp-ends complicated the stress distribution in the bed-joint to such an extreme that the general relationship assumed by BS5628: part 1[16] that the strength of the wall is proportional to the square of its thickness could not be assumed. Finite element analysis demonstrated that the stress-distribution is so highly modified by the presence of the perp-end that it can no longer be assumed linear. These findings have also been expressed by Anderson [27] and Drysdale and Hamid[30] who suggest that the section modulus methods of calculating stress may be inaccurate.

### 2.1.3 The Bond Failure Mechanism

De Vekey[31] acknowledged that the current British Standard BS5628: Part 1[16] identifies a relationship between flexural strength of the wallette and the mortar designation, which, to an extent implies that a relationship between flexural strength and compressive strength exists. Upon investigation De Vekey found that for bending parallel to the bed joint results were statistically significant whereas for bending perpendicular to the bed joint no correlation was found. The relationship between mortar flexural strength and compressive strength is considered to be a function of either cement content of the mix or water-cement ratio. Workers have given limited consideration to the possibility that the compressive characteristics of the mortar actually dictate the failure mechanism of the joint in flexural bending, more so than tensile bond properties. This is confirmed by Anderson's impression that the compressive behaviour of the mortar influences the flexural strength of the wallette.

Anderson[27] compared the cross couplet test with the type B wallette test outlined in BS5628[16] and found that strengths given by the wallette tests were higher than the couplet tests due, he suggested, to the stiffness of the mortar joint interfaces in tension being less than in compression. Anderson argues that the assumption that the tensile and compressive moduli of the mortar bed are the same in both tension and compression is not correct, since the area subjected to tensile stress will be greater than predicted by the normal assumptions. Essentially, this infers that the position of the neutral axis is not at mid-depth of the joint and its position can change during testing due to the compressive and tensile properties of the mortar.

Drysdale and Gazzola[32] indicate that there is no solid basis for relating tensile bond strength to the compressive strength of the mortar and they argue that it is unlikely that a unique relationship exists. This assumption was also refuted by Adams and Hobbs[33], who compared the 28-day mortar compressive strength with crossed-couplet bond and wallette bond and found that no clear relationship existed.

Baker[34] highlighted that in the direct tensile test an ultimate load is divided by an area, whereas flexural tests use a moment divided by a section modulus. Baker argues that shrinkage cracks, which occur in the extreme fibre of the joint, which are neglected in the calculation, reduce the section modulus significantly more than the corresponding area reduction. Baker estimates that if these shrinkage cracks extend 5mm in from the surface of the bed joint, for a normal sized stack bonded pier, an apparent reduction in strength of about 28% would be measured. This could possibly provide an explanation why flexural tensile tests yield higher bond strength results when compared to results obtained from direct tensile bond tests.

Isberner[35] found that cement hydration in the outer portion of the joint continues only while the relative humidity of the mortar exceeds 85%. He determined that the relative humidity in the outer portion of the joint, which he described as one-eighth the brick width, reduced to 80% after three days curing at 28°C and 50% relative humidity. Hence, hydration of the cement in the outer portion of the joint ceased after 3-days. Similar measurements showed that hydration at quarter width and centre of the mortar bed underwent 12-days and 15-days hydration, respectively.

Working on Isbener's interpretation that the extent of hydration varies across the joint width, the compressive strength properties of the mortar will also vary, further complicating assessment of flexural strength based upon the accepted bending theory.

A further disparity between the joint periphery and the body of mortar within the bed joint, is that any influence of carbonation will take place in the extreme edge of the mortar bed, due to the presence of carbon dioxide in the air. Consequently, the flexural tension approach, which is largely determined by the strength of the outer portion of the mortar joint, is further complicated by joint boundary conditions.

### 2.1.4 Comparison of Test Methods

Anderson[27] found that the ratio of crossed-couplets to all joints tested in a wallette was approximately 0.51 for brick samples. Adams and Hobbs[33] showed that the wallette tests gave bond strength up to 50% higher than the couplet tests. This value is similar to that found by Anderson, despite the work using wallette results rather than a mean of all joints, which, as discussed above, can yield slightly higher strengths. Adams and Hobbs also reported coefficients of variation for wallette flexural strengths between 10% and 48% and for crossed couplet bond from 9% to 50%; the average coefficient of variation being 24% to 27% respectively.

The above study of flexural testing techniques indicates that the bed joint periphery is more sensitive to flexural tensile bond than to direct tensile measurement. Because flexural and direct tension test approaches have different boundary conditions and different stress fields under loading, no realistic correlation can exist between the two.

Authors have questioned the methodology behind the crossed-couplet test, particularly in the joint formation process. For example, Anderson and Held[36] determined that more consistent results were obtained if the height through which the compaction weight dropped was doubled from 38mm to 75mm. This was also found necessary by Adams and Hobbs[33]. However, neither of these workers question the judgement of using a hammer drop to form the mortar joint. It is the consideration by the author of this work that although bricklayers may tap bricks to line and level in practice, this is achieved using several small taps, permitting the mortar to flow in a plastic state. A single drop however may cause a plastic mortar to change its retention properties due to the hydraulic pressure affects, which could ultimately influence the bond strength development. Notwithstanding, Pearson[37] found that the crossed-couplet test appeared to offer the most satisfactory test of the bond strength between the brick and the mortar. Adams and Hobbs also concluded that the cross-couplet test as a method of determining bond strength is preferable to the flexural wallette test.

A numerical evaluation of bond tests was performed using finite element analysis by Pluijm[38] in 1995. Pluijm concluded that the crossed-couplet test is not suitable for determining tensile bond strength and quality control since theoretically the test underestimates the bond strength by 43-53%. Pluijm claims that the direct tensile approach adopted in his analysis gave almost ideal results with a uniform stress field generated in the mortar.

Anderson and Held[36] determined during tests comparing wallette's and crossed-brick couplets that specimen format could influence results due to curing conditions. They reported a difficulty when covering crossed-couplets since there was more proportion of air to mortar, due to the shape of the couplet.

Adams and Hobbs[33] reported that the size of the test specimens required mortar with different levels of consistence, with higher initial dropping ball range required for manufacture of wallettes, in order to compensate for workability loss which occurred during the mortar's one to two hour usage.

Some workers, for example Palmer and Parsons[39], McBurney, Copeland and Brink[40], Thornton[41] , Kampf[42] and Ritchie and Plewes[43] have used water penetration techniques to assess the quality of bond at the brick-mortar interface by measuring the progression of moisture through the joint. Although this method serves to demonstrate that the integrity of the bond is an important factor in resisting moisture ingress, it is unable to quantify bond strength results in terms of an ultimate bond stress.

### 2.1.5 Summary of Literature

Flexural methods of determining tensile bond strength provide acceptable representation of lateral forces imposed upon single leaf masonry; quantitative results for different mortar and unit combinations can therefore be used for structural design purposes with suitable factors of safety being applied to account for the presence of uncontrollable variation.

It is considered by the author that flexural testing cannot be used for comparative research intended to distinguish between the effects of two or more treatments, due to both the nature of testing practice, specimen format and manufacture process.

Flexural testing, which adopts simple beam theory in its analysis, focuses upon the extreme fibre in tension and therefore is predominantly dependent upon parameters influencing the joint edge such as shrinkage, hydration and carbonation. These influences may not necessarily be representative of the overall bond. Furthermore these properties are not consistent and could be present as a result of the particular treatment under investigation. In addition, the presence of perp-ends and properties of the mortar in compression as well as tension can complicate the analysis used to determine flexural results and simplification of bending theory is not characteristic of true tensile bond strength.

It is concluded that both direct and flexural methods of tensile testing have their merits, however they are entirely different applications and cannot be correlated. Flexural methods are suitable for investigating structural aspects of masonry but are not sufficiently sensitive enough to investigate the nature of the bond formation processes. It is therefore imperative when discussing testing techniques that a clear distinction is made between direct tensile bond testing and flexural bond testing.

It is further recognised that there would appear to be no unique relationship between the compressive strength of the mortar and the tensile bond strength. Use of flexural bending test methods and associated theory has possibly compounded this misinterpretation. It is recognised and demonstrated in Chapter 3 that parameters preferential to both the compressive, tensile and bond behaviour of the mortar could be present but that any interaction is primarily coincidental.

## 2.2 Methodology of Tensile Testing Brick-Mortar Interfaces

### 2.2.1 Comparison of Direct Tensile Test Methods

The testing procedure adopted for this research was initially developed at Sheffield Hallam University by Taylor-Firth and Taylor[25], shown in and Plate 2.1, and has been subsequently modified as shown in Figure 2.1a) and b). The chosen methodology, for reasons explained below, is considered to have certain advantages over the ASTM crossed-couplet test, which at present is the only recognised form of direct tensile test.

While the actual mechanism of testing does not necessarily need to represent realistic loading criteria for comparative studies, joint formation does need to simulate true workmanship practice if the results are to be deemed representative of masonry. It should be recognised that workmanship parameters differ considerably between laying continuous courses and producing isolated couplet joints and detailed consideration of the joint formation process is important when choosing a testing methodology.

The couplet manufacture of the Sheffield Test compared to the ASTM crossed-couplet test differs in two respects. Firstly, the crossed-couplet test joint is formed by the operator who judges the correct quantity of mortar to use. The Sheffield test uses gauging bars to create a consistent, wet laid and compacted finished mortar bed. Secondly the crossed-couplet test uses impact loading to compact the bed joint, whereas the Sheffield test uses joint depth bars which permit the operator to generate a parallel, 10mm deep joint by a shearing action. It is believed that impact compaction of the crossed-couplet method may not be of sufficient duration to fully compress the joint due to the hydraulic nature of wet mortar.

Essentially the application of direct tensile force is difficult to implement since any misalignment in the test system will result in an eccentricity, inducing horizontal and rotational forces. Adjustment of the hanger bars in the Sheffield rig allows for dimensional imperfections in brick units to be accommodated. The load is applied at the bonding interface and therefore axiality is solely dependent upon the flatness of the bed face and remains unaffected by overall brick geometry. Samples are levelled and suspended by gravity, facilitating alignment with the direction of loading.

Furthermore, the application of axial loading may not necessarily be reflected by a uniform stress field at the brick-mortar interface due to flexing within the masonry unit. In order to minimise misalignment and variation in stress distribution the point of load application must be as near as possible to the interface under test. This raises the complication of fitting loading arrangements into the joint which in most cases is limited to a depth of 10mm and an area of approximately 215mm x 102.5mm. Encroachment into the joint will reduce the bond area and limit representation.

The adopted test has the advantage over the ASTM Crossed-couplet test[24] since the bond area is approximately 50% greater. The lever arm from the loading arrangement to the interface bond is of a similar length to the Crossed-couplet test.

The Crossed- brick couplet test relies on a compression loading to induce tensile bond stress. It is argued by Taylor-Firth and Taylor[25] that the failure load recorded by such a force will be in excess of the true tensile force required to separate the samples due to the response of the test system. The Sheffield Test uses tensile force, which is continuously traced by an integrated calibrated plotter, which retains a graphical record of true peak load.

### 2.2.2 Manufacture of Brick-Mortar Couplets

For the majority of the sample preparation in this thesis, Fletton brick units (refer to Chapter 3 for details) were bonded using their plain bed-face to eliminate the influence of frogs or perforations upon bond strength formation and measurement. The joint forming jig shown in Plate 2.2 has two gauging bars, which slide up and down normal to the bed plane, to accommodate variation in unit size. The lower brick is placed in position, plane face uppermost and the gauging bars rotated to lie perpendicular across the bed face. The gauging bars are 15mm deep and allow a wet mortar bed to be levelled to an initial depth of 15mm and length 150mm.

A portion of mortar is then taken from the spot-board and turned over three times to impart cohesivity to the mortar before being firmly “thrown” onto the brick between the gauging bars. A 225mm gauging trowel carries sufficient mortar to form a 150mm long by 100mm wide by 15mm deep, wet mortar bed. The trowel is then used to level and compress the mortar to 15mm depth. The width of the mortar bed is determined by the width of unit since mortar is struck-off level with both faces of the unit. The gauging bars are then rotated away to release the lower brick.

The lower brick is then immediately positioned in a second jig, which is equipped with 10mm joint depth roller bars, as shown in Plate 2.3. The brick is centralised and its forward face placed up against alignment blocks. The upper brick, stretcher face forward, is then placed onto the mortar bed and pushed with a shearing action until its face meets the alignment blocks and its ends are level with the lower brick. The 10mm joint depth roller bars, which restrict further compression of the joint, are then removed simultaneously using a rotating action, in a direction of pull normal to the stretcher face of the couplet.

The sample is removed from the jig and any excess mortar which may have been squeezed out of the joint struck-off flush with the stretcher face. The samples are allowed to stabilise before tooling of the joints, which are pointed on completion of every third couplet, corresponding to approximately 15-minutes of mortar spot-board life. Samples are then cured in the laboratory (20°C, 40% R.H.) for up to one hour in order to ensure that sufficient strength had been gained to avoid damage while moving. Samples are then transferred to a curing chamber (20°C, 80% RH) to await testing. For the majority of work samples have been tested 28-days after manufacture unless stated otherwise.

### 2.2.3 Description of Tensile Test Apparatus

The tensile test rig was designed to be compatible with any standard tensile testing machine including Houndsfield Tensometers. The rig consists of two similar but not identical loading units. Each unit has a female connector, which locates onto the male connector of the test rig and is held in position by a pin.

A mild steel rectangular bearing plate is supported at its centroid by the connector which is fixed by a bolt on the underside of the plate. The bearing plate has a profiled section, which has been designed such that the section modulus is consistent with the theoretical bending moment profile. The bearing plate supports four 6-mm diameter mild steel rods via a rotational bearing at each corner. These are located through holes and seated by a cup and cone type bearing. The four hanger rods, two at each end, support a loading bar which is seated using a similar cup and cone type bearing. The hanger rods are threaded at one end to allow the loading bars to be levelled in the horizontal plane.

The standard couplet joint depth is limited to 10mm and therefore there is no facility to position the upper and lower loading bars symmetrically in the bed joint, since the loading bars may potentially carry considerable load and need to be of section depth greater than half the joint depth to resist bending. Consequently the centres of the lower loading bars are offset by 11mm towards the end of the bearing plate. The couplet joint depth cannot be increased since the sample joints need to be representative of masonry. Hendry[44] has shown that excessively thick bed joints of between 16-19mm may be expected to reduce the strength of the brickwork by the order of 30% compared to a normal 10mm deep joints. This view is also supported by Sise, Shrive and Jessop[45] who report on the tremendous change in bond strength with joint thickness over the range 6 to 15mm.

### 2.2.4 Modification of Test Apparatus

The original loading bars shown in Plate 2.1, supported the sample on knife-edges across the brick width. Due to brick dimensional tolerances being poor, it was found that the bed-face was not contacting at all points along the length of the knife-edge but rather was bearing at discrete points which could vary between samples and more importantly vary between loading bars for a particular sample. The screw threads on the hanger rods are tightened so that the sample is held firmly by the loading bars within the joint and therefore it was considered that the alignment of the bars could be affected by any slight undulation in the brick surface and hence impart eccentricity to the test system. The problem was overcome by using raised contact points, which provided a two point loading system on each bar and which supported the sample at four specific points, both top and bottom; refer to Figure 2.1a) and b). The raised contact points needed to be of sufficient depth to exceed any deviation in bed-plane flatness. By incorporating these discrete points of contact in the design, the section modulus of the loading bars had to be reduced to enable the spreader bar to be located within the 10mm bed-joint depth. The depth of the loading bar was reduced from a maximum of 8mm at the knife-edge to a flat bar of 5mm thickness to allow for 3mm deep disks to be used as points of contact. It was found that the reduction in section of the loading bars dramatically reduced the section modulus of the bar and could cause the bar to deflect within the depth of the joint during sample testing. Bending of the loading bar would impart serious moment into the hanger bars and more importantly, if the deflection exceeded the clearance of the bar within the joint, cause the bond to fail prematurely.

At the predicted working load of the test system, it was calculated that the deflection in the hanger bars would exceed the joint clearance if the depth of the section was reduced to 5mm. To allow for this phenomenon the elastic modulus of the loading bars had to be increased.

The loading bars were heat treated at 820°C and then cooled in oil. The steel was then tempered at 200°C and finished at 58 Rockwell. This treatment gave an increased modulus of elasticity from 210 $kN/mm^2$ to 320 $kN/mm^2$. Two 3mm deep disks, 5mm in diameter were then spot-welded to the loading bars at 75mm centres. A calculation based upon the new modulus of elasticity confirmed that deflection induced under ultimate bond strength loading, would not exceed the joint clearance.

### 2.2.5 Testing Procedure

The top four hanger rod screw threads were adjusted in order to level the two top loading bars with the aid of a 10mm thick toughened glass plate and spirit level. This ensured that the top plane of the test rig was always horizontal, irrespective of any deformity within the joint or brick bed-face. The couplet was then suspended by the upper unit seated on four raised points of contact close to each corner. The lower bars, which are positioned outside the upper bars were then manoeuvred into position. Verticality of all hanger rods was checked visually in both planes before tightening the lower adjusting screws; the adjusting screws were adjusted until finger tight by tightening diagonally opposite bars in rotation.

A tensile force was applied through a J.J. Lloyd Testing Machine fitted with a 20kN load cell, at a loading rate of 5mm/minute. The applied load was recorded using an electronic pen plotter, which recorded deflection along the abscissa and load on the ordinate axis. The plot was then measured to obtain the ultimate failure load; 1mm deflection on the graph corresponding to 20 Newton's at a load factor of 0.4. The load cell and plotter were independently calibrated, as reported in section 2.4.1. The bond area and plane of failure were recorded on the chart after each sample was tested, along with treatment type, mortar volume ratio and water content, sample manufacture date and intended test date.

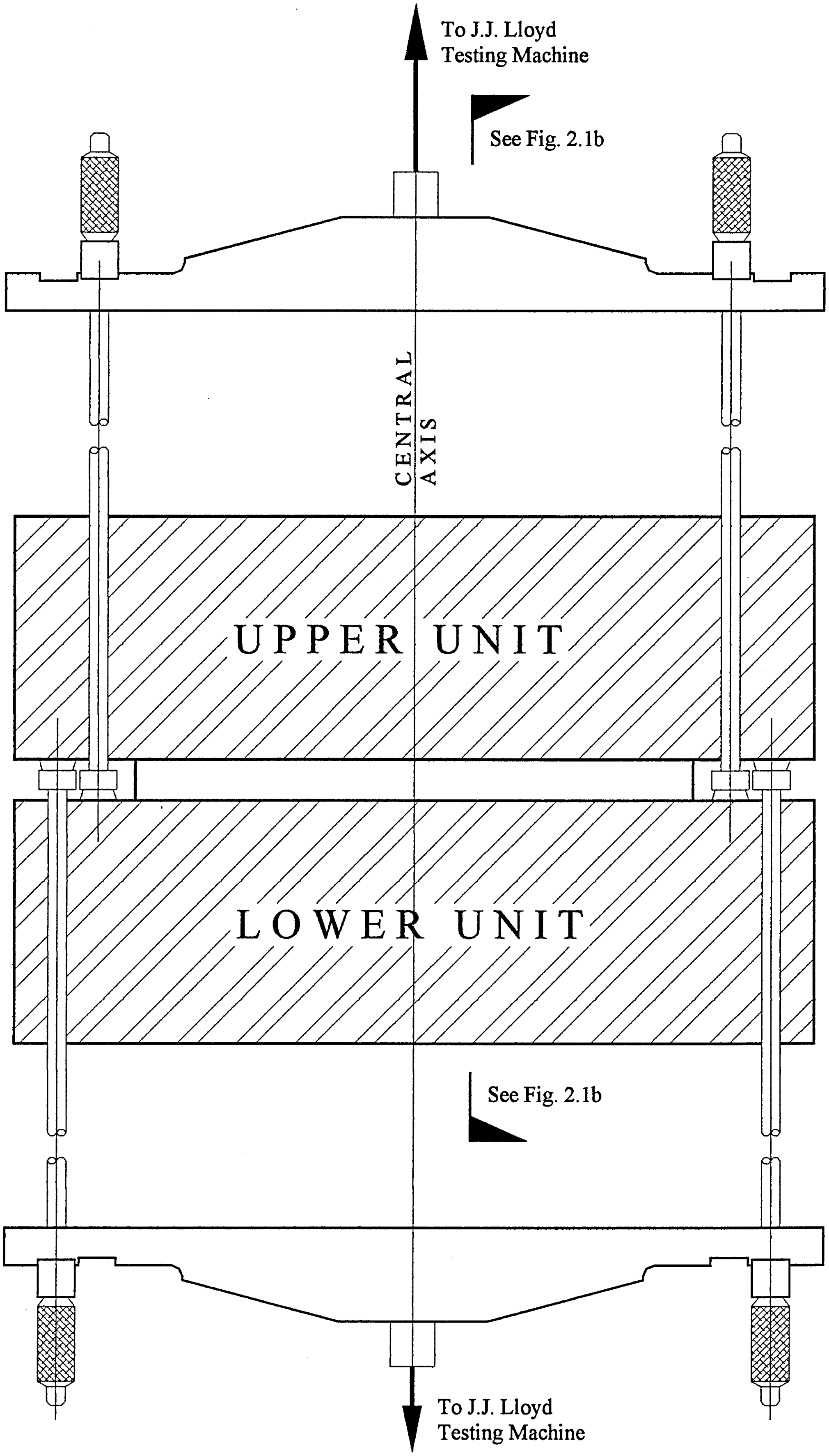



**Figure 2.1a : Side Elevation on Sheffield Hallam University Tensile Test Rig** (not to scale)

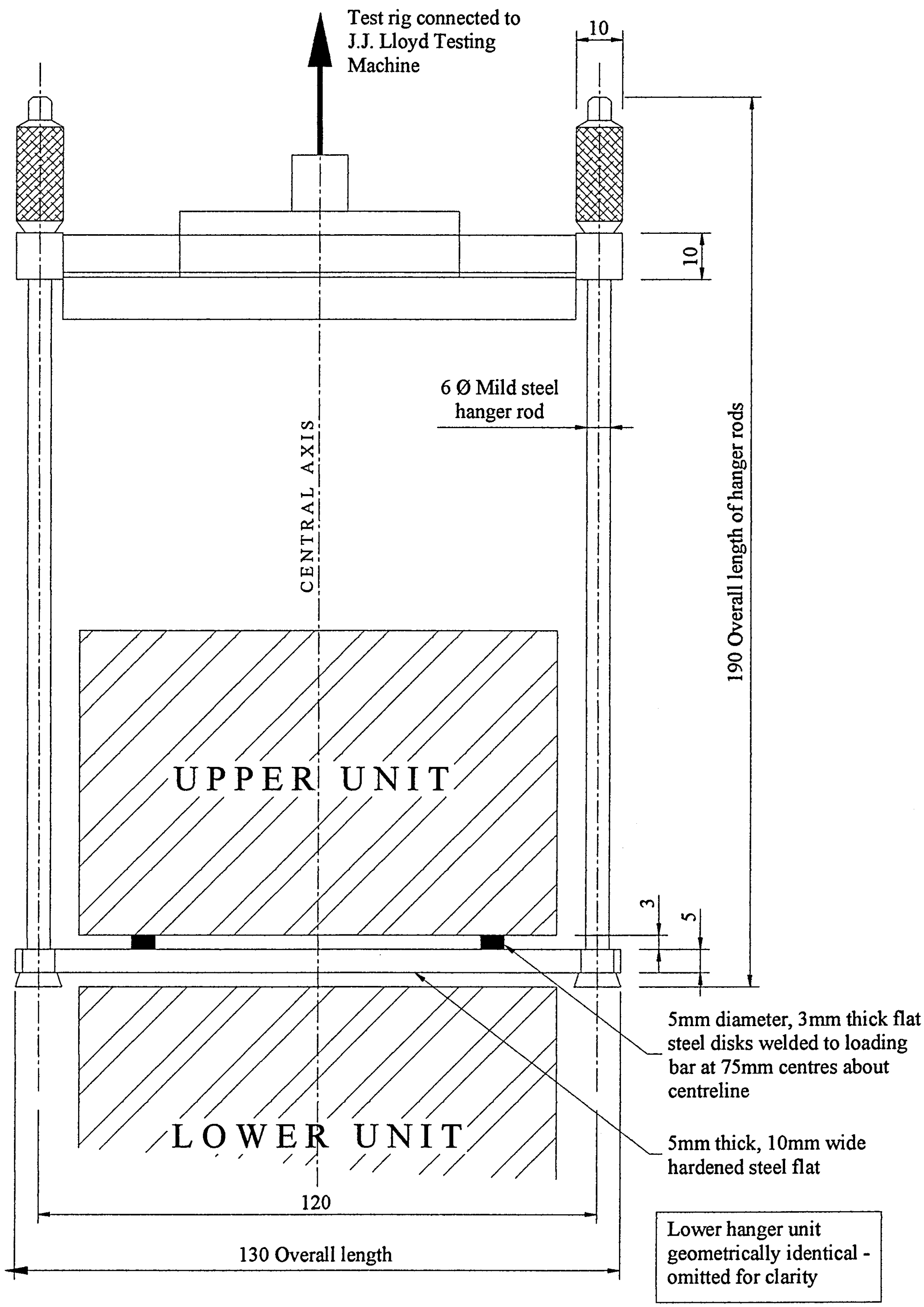



**Figure 2.1b : Sectional Elevation Through Upper Hanger Assembly** (not to scale)

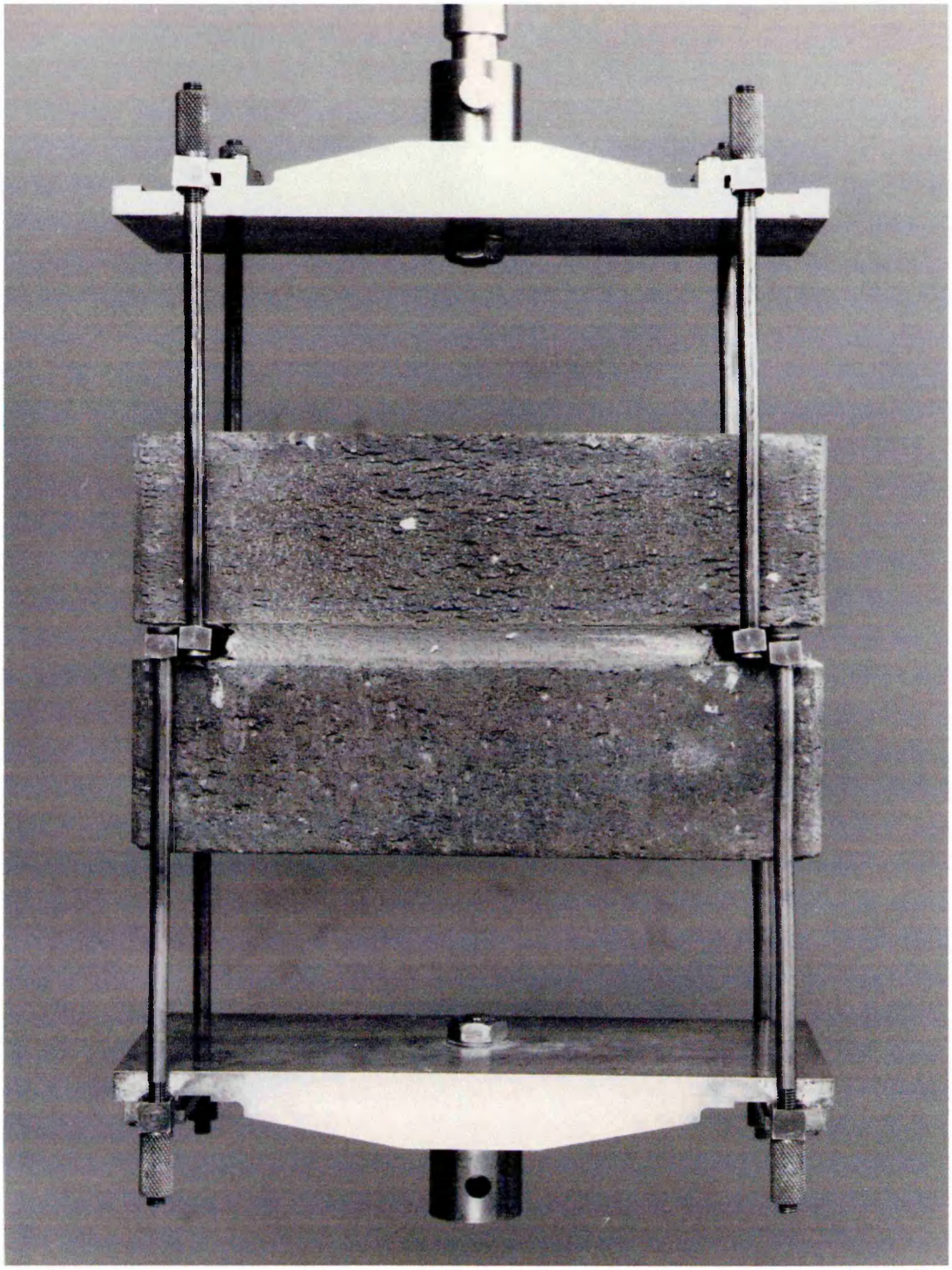


**Plate 2.1: Showing Side Elevation View on Sheffield Hallam University Tensile Test Rig Before Modification**

**Plate 2.2: Showing apparatus for making 15mm thick mortar bed**

**Plate 2.3: Showing couplet joint forming apparatus**

## 2.3 Experimental Verification of Direct Tensile Methodology

### 2.3.1 Strain Monitoring of the Testing Apparatus

To establish the validity of the test apparatus for imposing direct tension and therefore measuring tensile bond strength of brick-mortar interfaces, it was required to investigate the distribution of load in the test apparatus. Measurement of the strains induced in opposite faces of each hanger rod were taken, in order to identify the presence of bending in the hanger rods. An average of the strains in opposite faces of the rods would compensate for the effects of bending upon strain results and indicate the magnitude of overall tensile strain in the rod. Proportion of distribution of tensile strain between the hanger rods would thus confirm axiality of the test approach.

Originally, 20mm long resistance strain gauges were positioned on the outside face of each rod at mid-length. It was found however that in order to get a true assessment of strain distribution in the hanger rods, resistance strain gauges would need to be fitted in pairs on opposite faces of the hanger bars to counter any bending influence. In addition 20mm gauges over a 160mm long rod tended to exaggerate the influence of end fixity of the bar.

Therefore 3mm foil strain gauges were adopted, positioned in pairs on opposite (180 degree) faces of the rods at mid-length.

Originally, strains were monitored for each load increment using a digital strain indicator with readings recorded manually for each of the strain gauges. Having positioned two gauges to each bar it became impractical to record 16 readings for every load increment, particularly for the model joints where there was a danger that the load would fluctuate as the joint began to fail. Consequently, strain was monitored continuously using a Solatron Orion 3530 Data Logger, which recorded strain in all gauges and load output from the load cell simultaneously.

In addition to the sixteen strain gauges on the hanger rods a further gauge was placed on a separate rod, not included in the test system, to compensate for the effects of temperature. Any compensation for temperature was carried out automatically via the data acquisition software.

Measurement was conducted to determine the strain distribution through the test apparatus during loading. Four different sample joint types were adopted, designed to exaggerate possible joint deformity of the couplet. It was considered that joint deformity could influence the axiality of the test system.

**Model joint 1**: No sample joint, upper hanger rods loaded against lower loading plate.

**Model joint 2:** A parallel plywood joint, 10mm bed joint depth.

**Model joint 3:** A plywood joint sloping across width from 10mm to 15mm.

**Model joint 4:** A plywood joint sloping along length from 10mm to 15mm.

The first model tests the axiality of the test apparatus with no model joint present. The original test rig was designed so that the upper or lower sets of hanger bars could be situated on the opposite loading plate, as shown in Plate 2.4. This approach permits an assessment of strain profiles in the rig, without the complication of bending within the test specimen.

The other three model joints were designed to investigate the influence of joint deformity upon strain distribution within the hanger bars. Each model was constructed from plywood and consisted of two 20mm thick plates of 215mm x 102mm plan area representing the brick. The mortar portion of the joint was again constructed from plywood, with the joint shaped to represent deformity and plan area 150mm x 102mm. The samples were then bonded using adhesive. Because the joint component carried the deformity, each model joint had one normal plane, which was located on the upper loading bars.

Plywood was adopted for the test specimens because it was considered important that the test sample had a lower elastic modulus than brick. Similarly, a 20mm thick plate was used to produce a reduced section modulus and therefore exaggerate the effects of any bending within the sample.

### 2.3.2 Procedure for Attaching Strain Gauges

Foil strain gauges were attached to the hanger bars using epoxy resin. Surface preparation for strain gauge bonding was achieved by degreasing the steel with isopropyl alcohol. The gauging area was then abraded using grit silicon-carbide paper, firstly with grade 220-320 and secondly with 400 paper. A drafting pencil was used to mark the alignment marks. The surface was then cleaned using acetone solution and cotton tipped applicators, wiping in a single direction each time. The gauge was removed from its acetate envelope by holding the edges of the backing with tweezers and placed bonding side down on a chemically clean glass plate. The solder terminal was positioned on the glass plate approximately 2mm from the wiring end. The foil gauge and solder contact were then picked-up by rolling mylar tape over the glass plate and the tape lifted from the glass at a shallow angle, lifting the components with it.

The gauge assembly was positioned with one end of the Mylar tape fixed to the bar, to align with the pencil marks. P6 adhesive was applied to both the gauge back and solder contact and gauging area on the bar using an applicator. The tape was then bridged over the gauge area at an angle of approximately 30 degrees; with a piece of gauze the tape was wiped onto the bar with a single stroke making sure no air became entrapped. Several wipes in one direction were made with the gauze to remove as much adhesive as possible from between the gauge and the bar surface. The bar was then held in the hand for two minutes to allow the body heat to cure the adhesive. The Mylar tape was then peeled away at a very shallow angle to reveal the gauge and solder contact.

The necessary wires were soldered to the solder block and lead wires attached. The bars were then shrink-wrapped in a plastic sleeve for protection.

When fixing strain gauges it is important to ensure that any pressure applied to the gauge during adhesion does not cause the bar to bend, since negative strain will be induced in the strain gauge upon straightening of the bar. Therefore, a special jig was used, with a groove to support the bar while pressure was applied. A line was marked at the end of each bar across the diameter to ensure that the bar could be rotated through exactly 180 degrees when attaching the gauge on the opposite face.

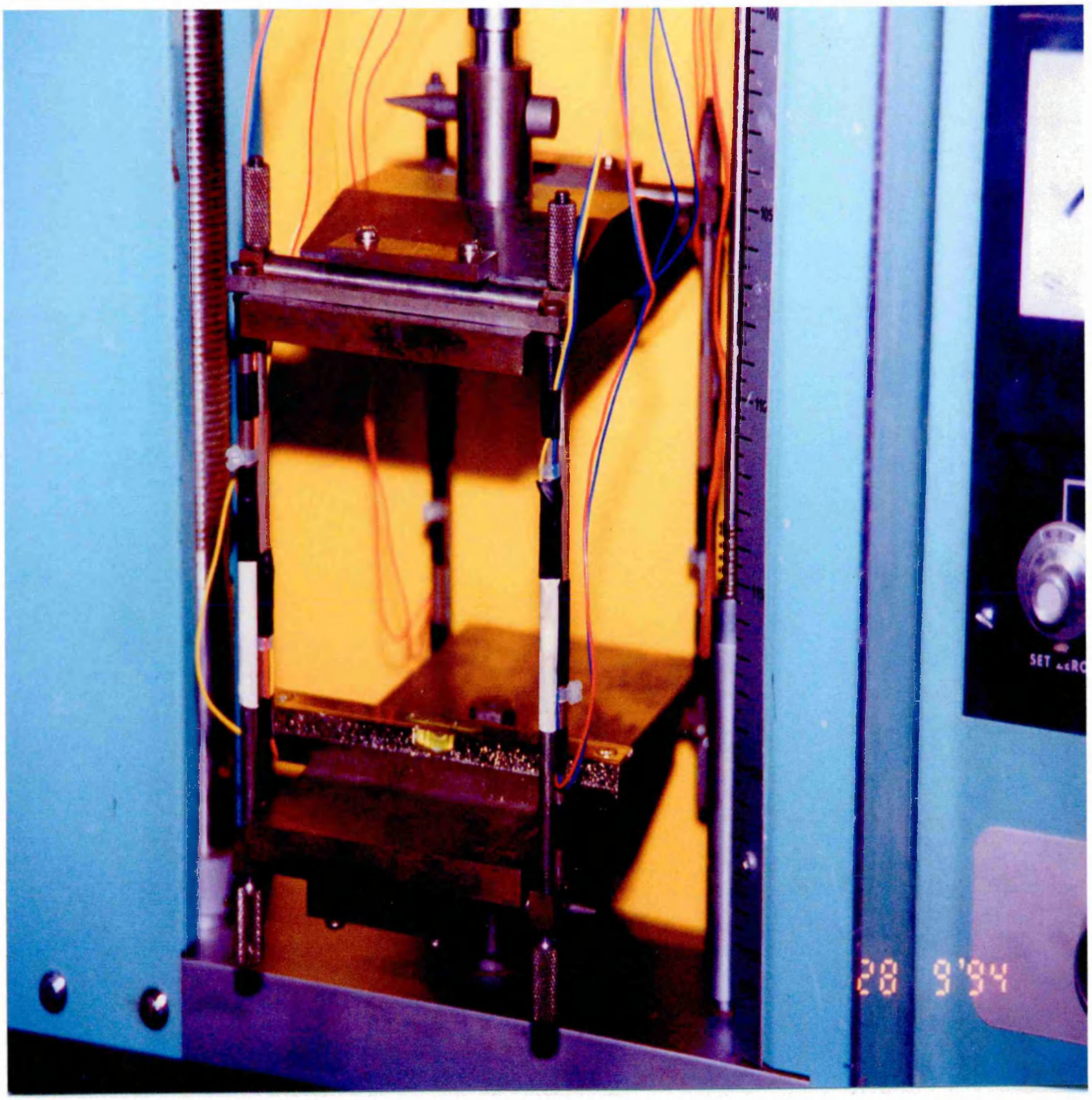



**Plate 2.4:** **Showing strain gauge monitoring on Tensile Test apparatus No sample joint; upper loading cradle loaded against lower loading plate**

## 2.4 Experimental Results

### 2.4.1 Calibration of Load Cell

Strain analysis of the test apparatus was designed to indicate the distribution of load in the test system. The final output for a masonry couplet produces a peak failure load, which is a function of both the load cell accuracy and the calibration of the pen plotter used to record the peak load.

At the time of the investigation, only a 20 kN load cell was available and concern was given that the working range of brick couplet bond testing was at the lower end of the load cell range. An average failure load for bond of a typical brick couplet is in the region of 2000 Newtons with perhaps 5000 Newtons being the upper bound limit for testing associated applications, such as the bonding of renders or enhanced bonding performance as a result of mortar additives. Ideally the working range of testing should have been at mid-scale of the load cell range. Therefore, if resources would have permitted a 5kN load cell should have been used.

However, it was considered that a 20 kN load cell is capable of measuring lower loads and the purpose of calibration was to ensure that a linear response over the loading range was obtained.

Several methods were used in an attempt to calibrate the load cell and plotter. The most straightforward method was to hang calibration weights on the load cell and monitor the deflection output from the plotter. However, due to the test rig geometry, there was a limit to the number of weights that could physically be suspended and therefore the anticipated working range of the load cell for testing brick couplets could not be tested for linearity.

The elastic modulus of a single mild steel bar with the same material and section properties as the hanger rods was calculated. The load cell bar was fitted with 3mm foil strain gauge in pairs as described in Section 1.3.2. The bar was removed from the J.J. Lloyd test apparatus and was held in tensile jaws used in a standard tensile testing procedure. The bar was then loaded in 5kg increments using calibration weights, the mass of each calibration weight having previously been independently determined. At each load increment the value of strain in the bar was recorded. The bar diameter and cross-sectional area were measured and a stress strain graph subsequently plotted; the gradient of which gave the elastic modulus of the bar.

Having determined the elastic modulus of the bar using standard weights, it was possible to use the same bar to calibrate the tensile test rig load-cell and pen plotter. The strain gauge and load cell output were monitored using the Solatron as Section 1.3.1. Several loading runs were conducted with the bar rotated in different positions in order to compensate for the effects of any bending present. At each loading increment the pen recorder marked a position on graph paper, this allowed for the independent calibration of both the load cell and plotter.

### 2.4.2 Strain Distribution

All sample joints were loaded to excess of 3kN, considered to be the upper limit of brick-mortar bond strengths. No strain profile is presented for a standard brick-mortar couplet because the failure load is comparatively low and the joint so brittle that a linear strain response was not presented. However, the purpose of the investigation was not to show strain characteristics of a masonry couplet but rather to demonstrate that the test rig was capable of imparting direct tensile force.

The recorded strain and loading results are shown in Figures 2.2 to 2.5, for each of the model joints described in section 2.3.1.

Figures 2.2 to 2.5 a) show the relationship between load exerted on the test specimen and the compressive and tensile strains in the hanger rods for each of the model joints.

Figures 2.2 to 2.5 b) show the average of the compressive and tensile strains in the hanger rods.

Figures 2.2 to 2.5 a) and b) show both the loading and unloading strain profiles. The maximum load differs for each model joint since each sample was loaded up to a point where bond failure in the joint was seen to be initiated.

The inlay on each chart shows the orientation of the test system and the direction of slope for those model joints with deformity. In all cases, the top loading bars were levelled horizontally and adjustment was made to the lower hanger bars in order to accommodate deviation from plane parallel.

Since the majority of strain is of a tensile nature, tension is shown as positive and compression as negative, contrary to convention.

The notation for inside/ outside refers to the strain gauge on that portion of the bar which is positioned either on the inside or outside of the test rig facing the test sample. The letters shown in the key refer to the positions of each hanger bar relative to the view from the side of the test machine, with the controls on the far side of the test apparatus. (**T**= top, **B**= bottom; **L**= left, **R**= right; **F**= front, **B**= back), in that order.

Figures 2.2 to 2.5 c) shows the percentage distribution of the total tensile load in the test system, carried by each of the hanger rods. The theoretical load is calculated using a previously determined value of Elastic Modulus of 210 $kN/mm^2$ for the hanger bar material (refer Section 2.4.1), the average tensile strain in the bar and an assumed cross-sectional area for the rod based upon a diameter of 6mm. The error value shown for Figure 2.2 to 2.5c) was calculated as the difference between the total theoretical stresses carried in each bar and the actual load recorded by the load cell. It is important to note that the error value is shown as positive in each chart. However in all cases other than for Joint Number 1 the summation of the theoretical stresses in each bar exceeded the load recorded by the load cell, yielding a negative value of error. The loading increments shown in Figure c) are approximately quartiles of the peak load applied to the model joint in question; no unloading profile is shown in this case.

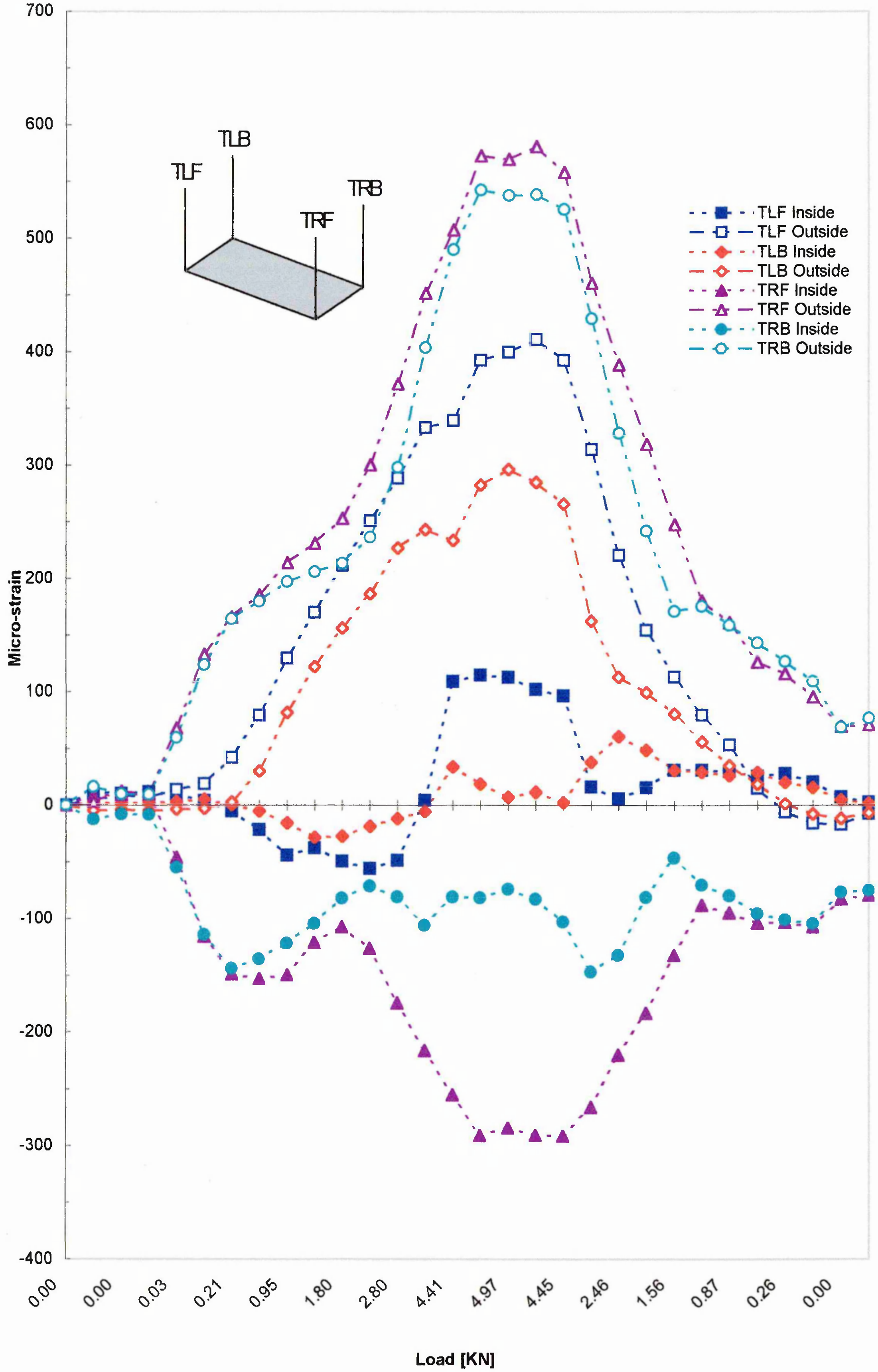



**Figure 2.2a Distribution of bending strain in top four hanger rods. Model joint 1, upper rods loaded against lower plate.**

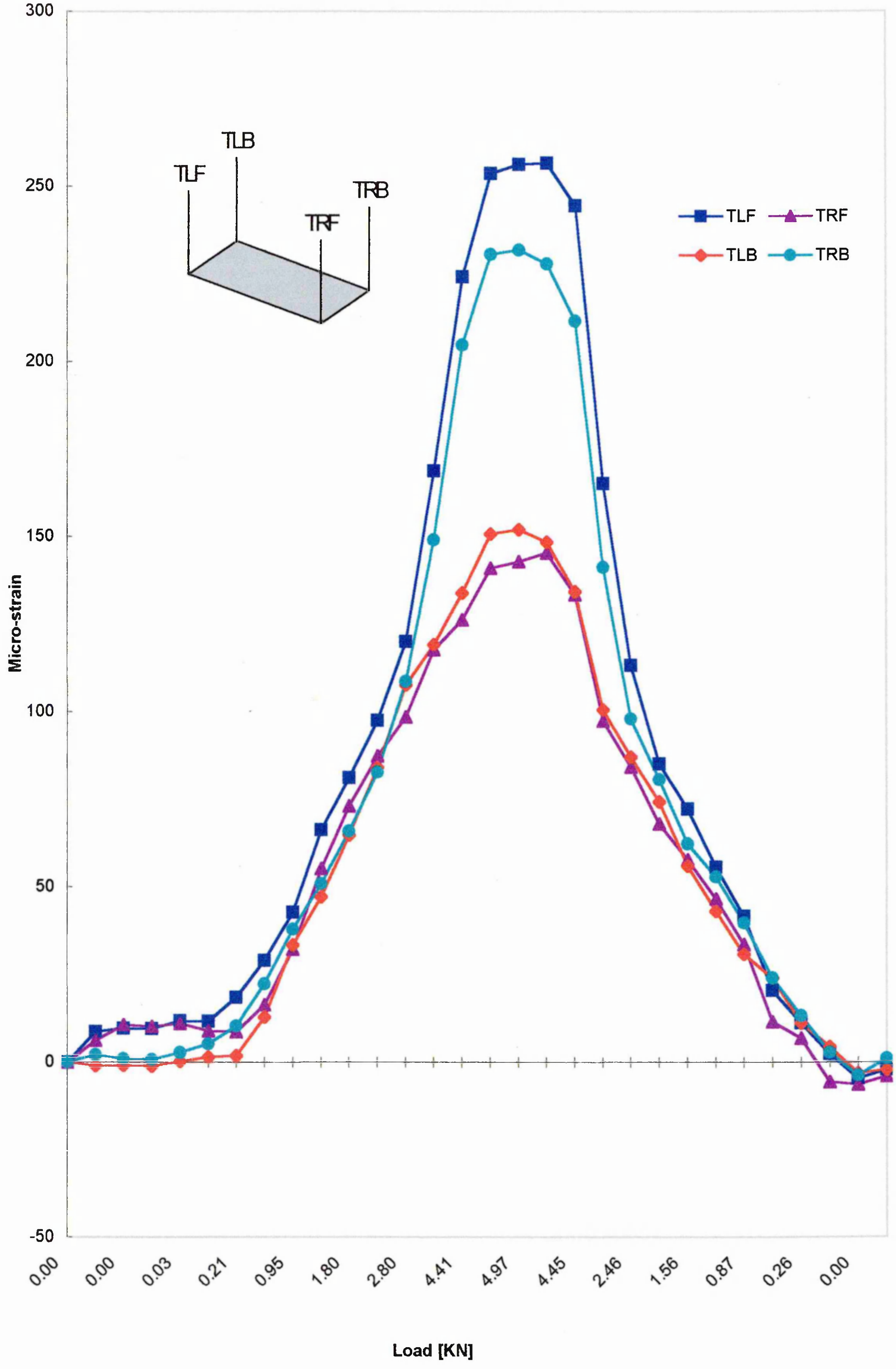



**Figure 2.2b Average distribution of strain in top four hanger rods. Model joint 1, upper rods loaded against lower plate.**

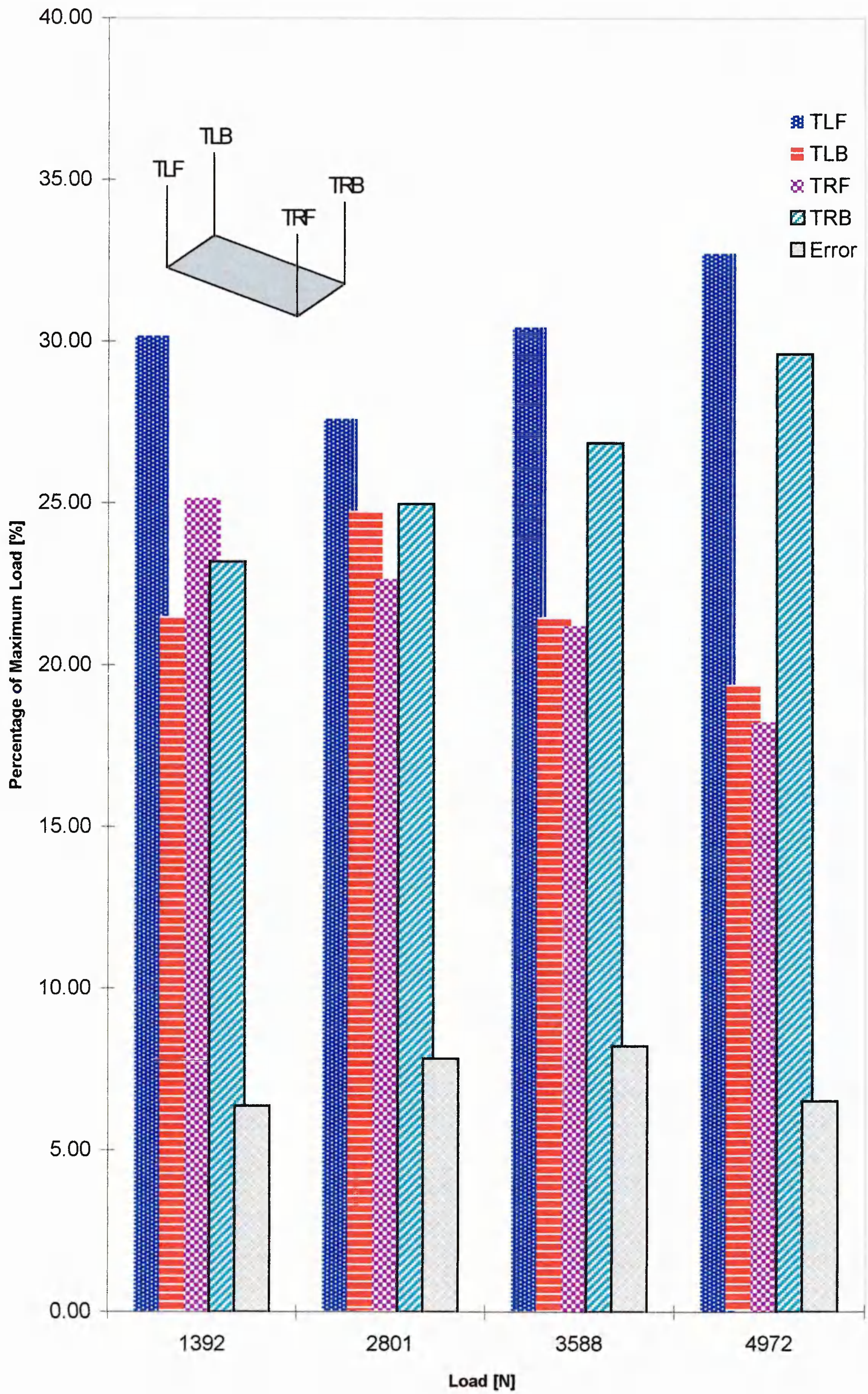



**Figure 2.2c Percentage of total load carried in each of the four top hanger rods. Model joint 1, upper rods loaded against lower plate.**

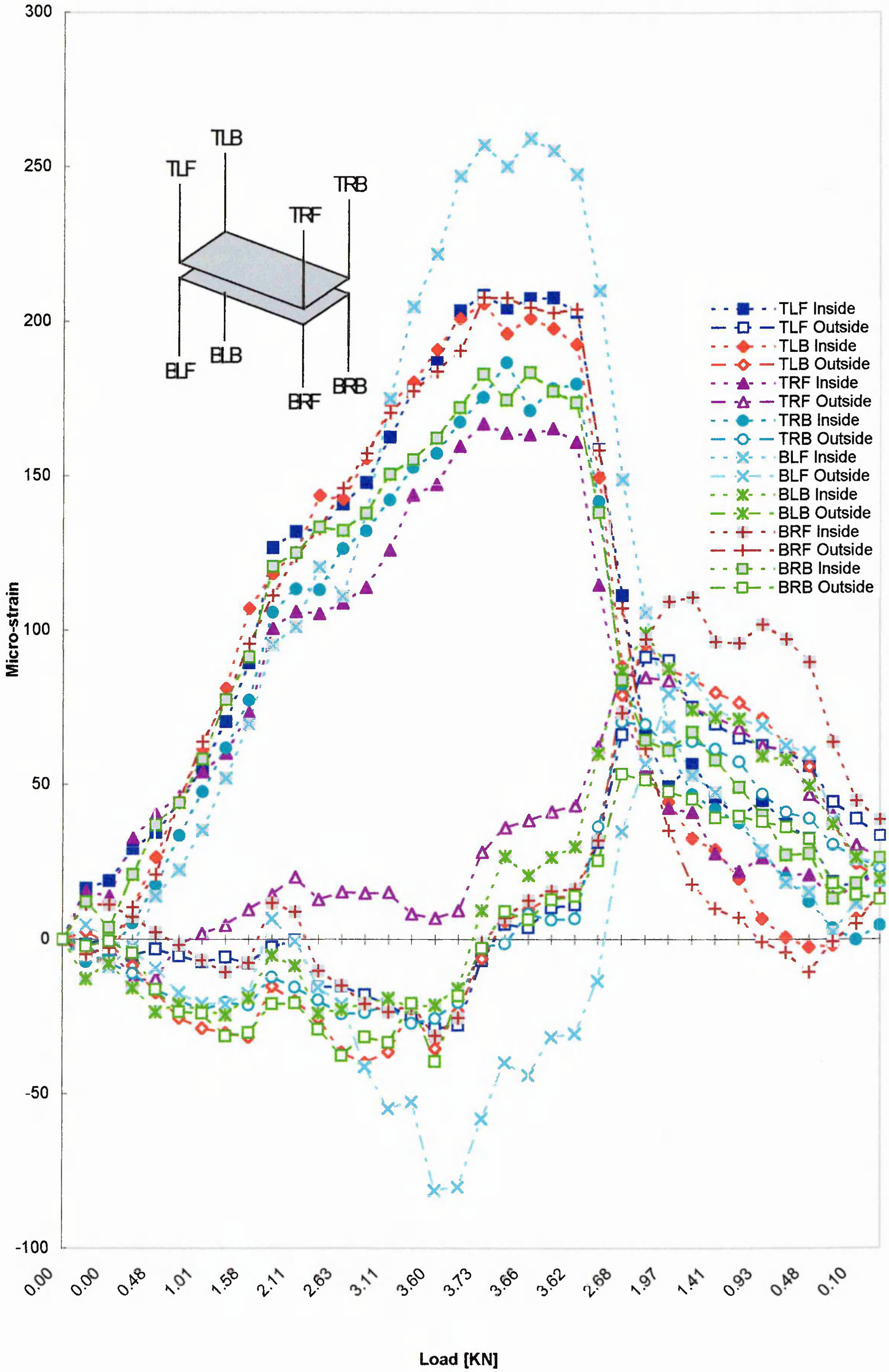



**Figure 2.3a Distribution of bending strain in all eight hanger rods. Model joint 2, parallel model joint, top plane horizontal.**

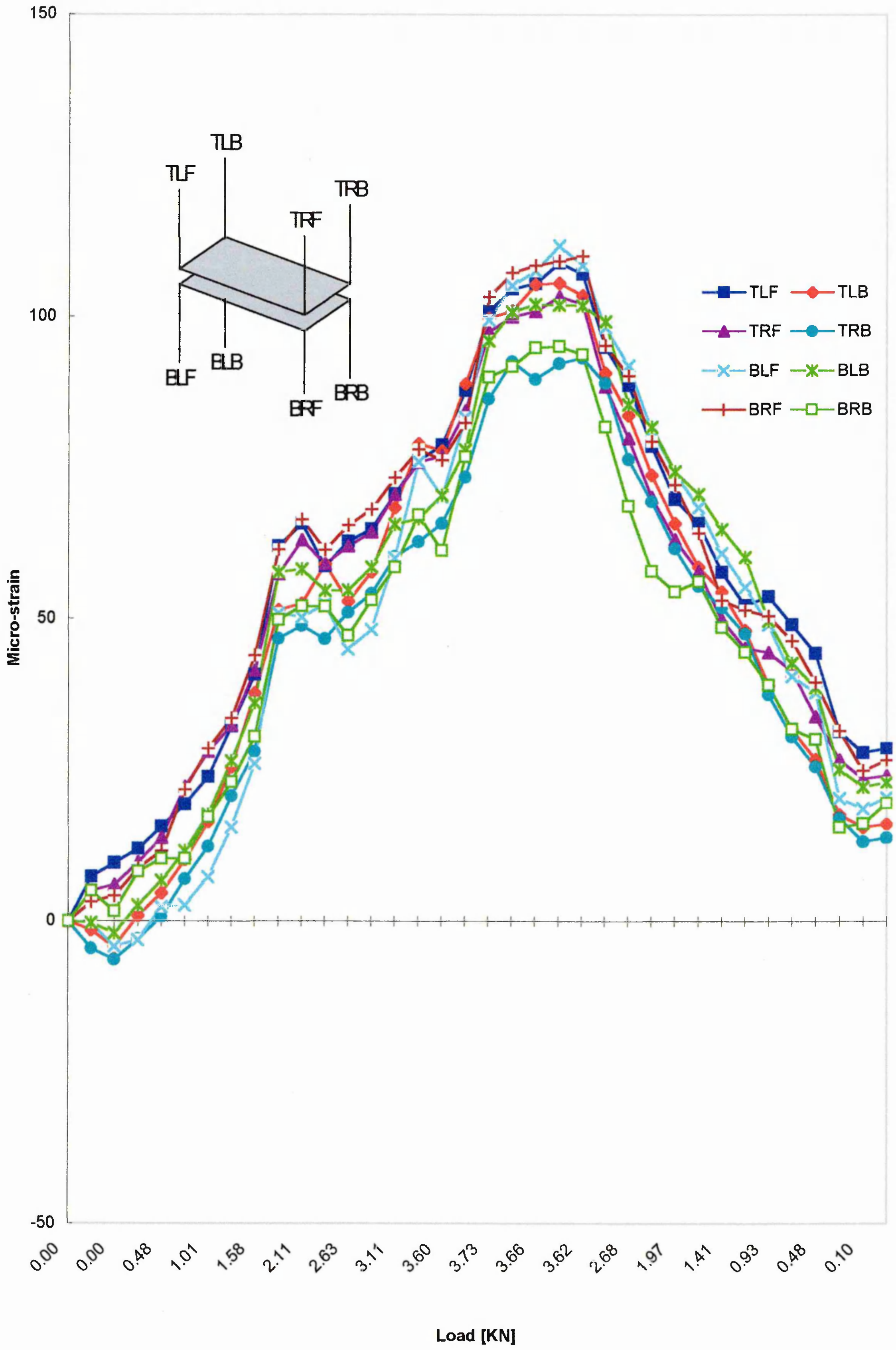



**Figure 2.3b Average distribution of strain in all eight hanger rods. Model joint 2, parallel model joint, top plane horizontal.**

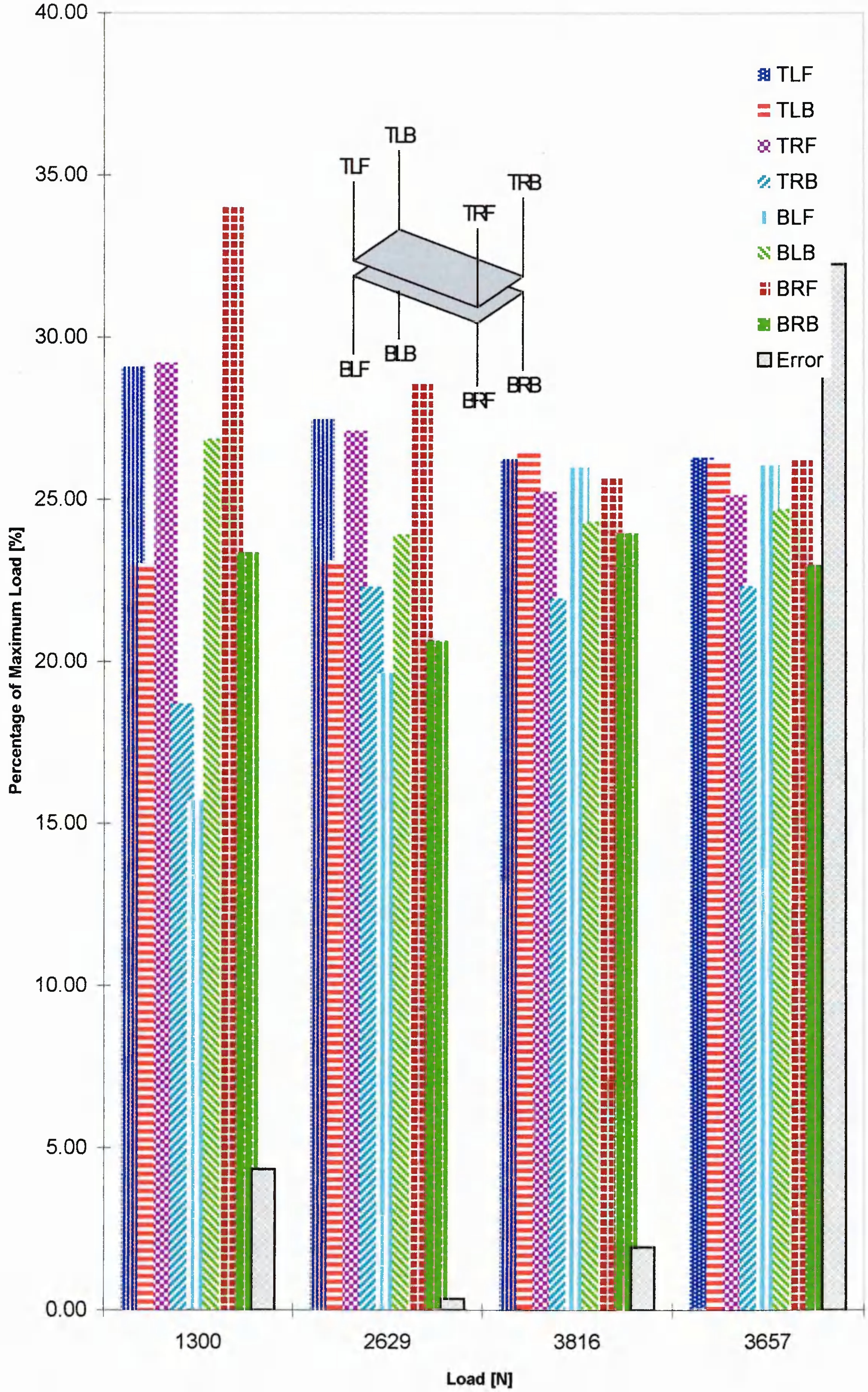



**Figure 2.3c Percentage of total load carried in each of the eight hanger rods. Model joint 2, parallel model joint, top plane horizontal.**

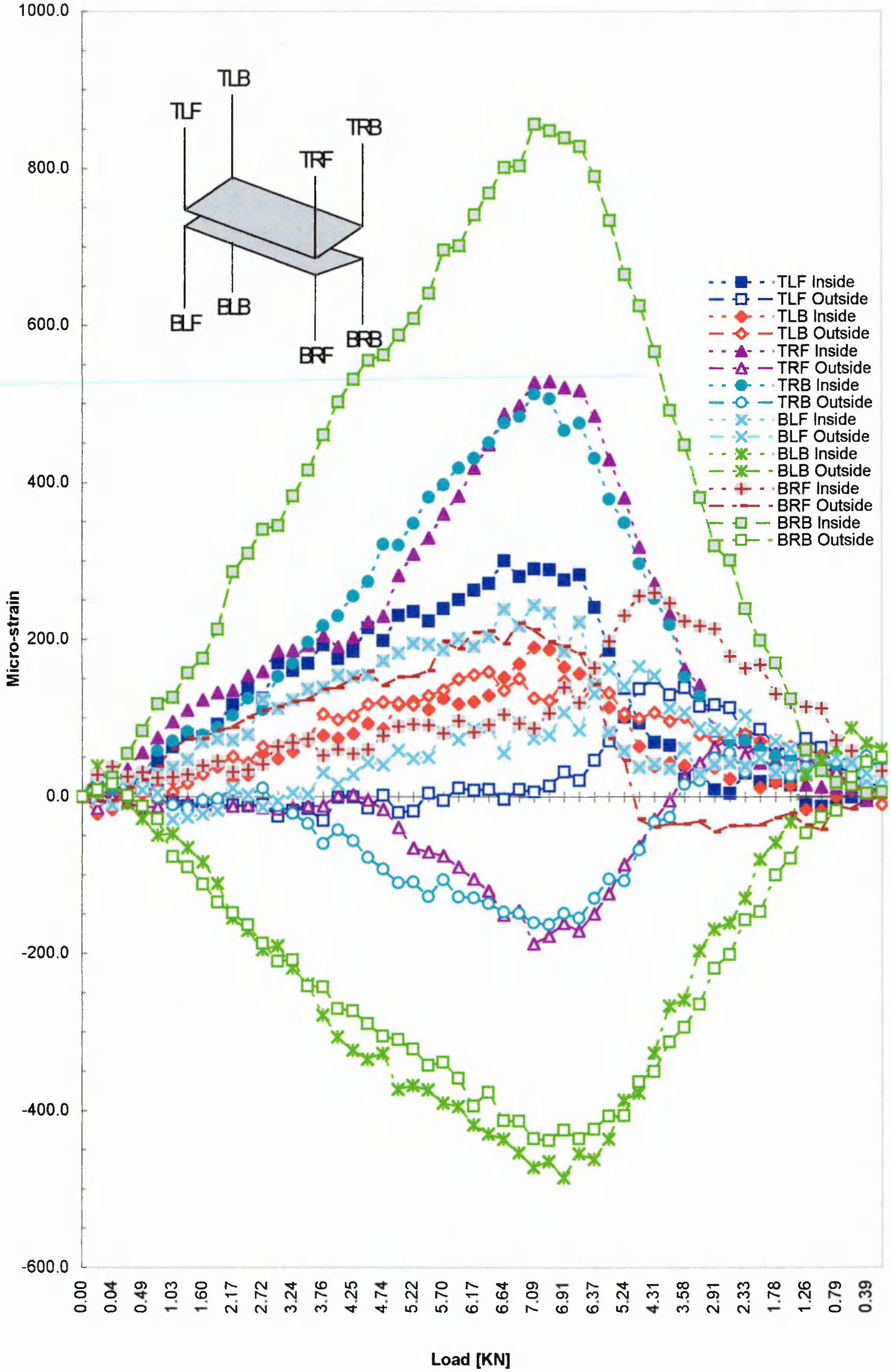



**Figure 2.4a Distribution of bending strain in all eight hanger rods. Model joint 3, sloping across width, top plane horizontal.**

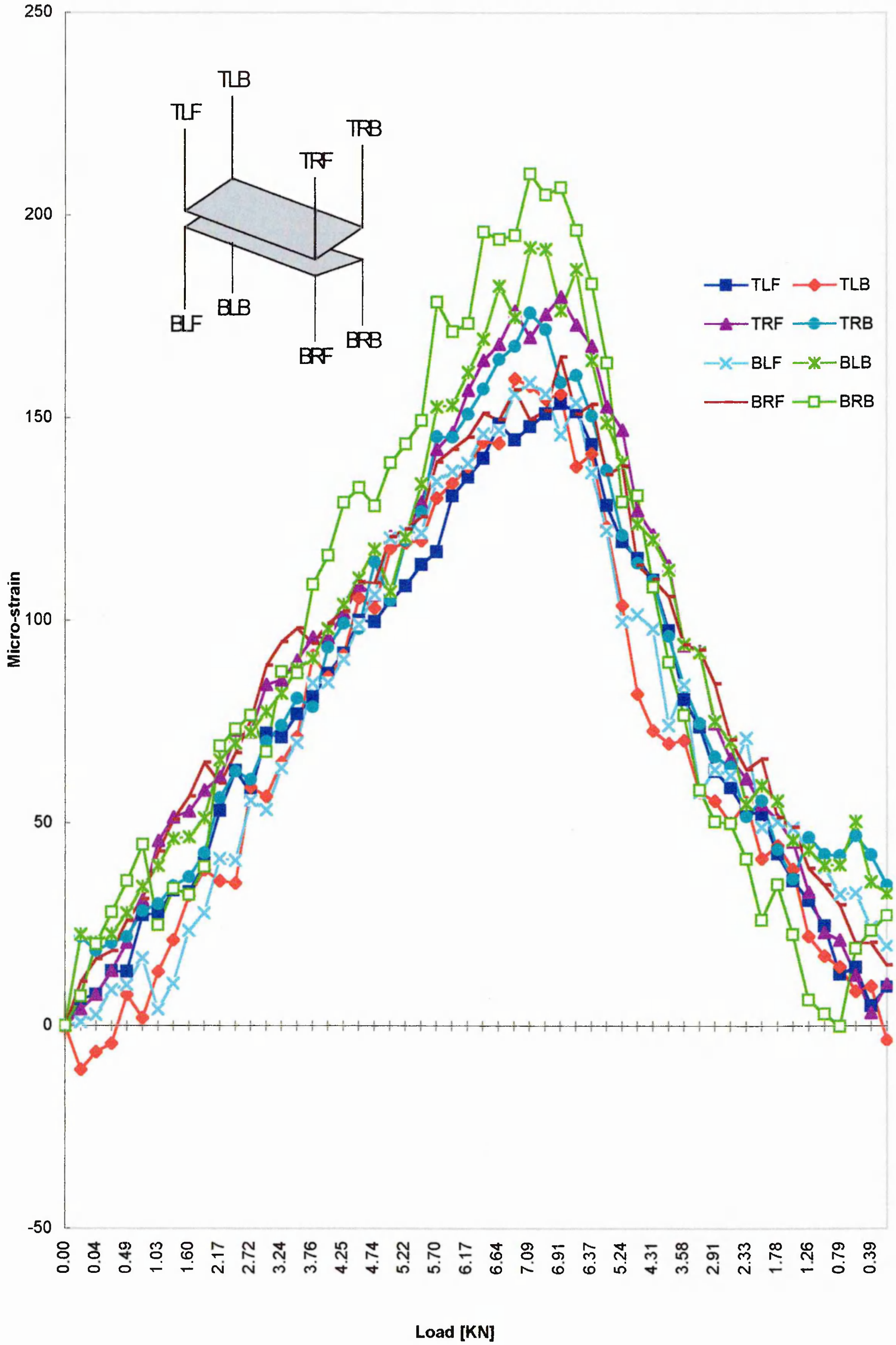



**Figure 2.4b Average distribution of strain in all eight hanger rods. Model joint 3, slope across width, top plane horizontal.**

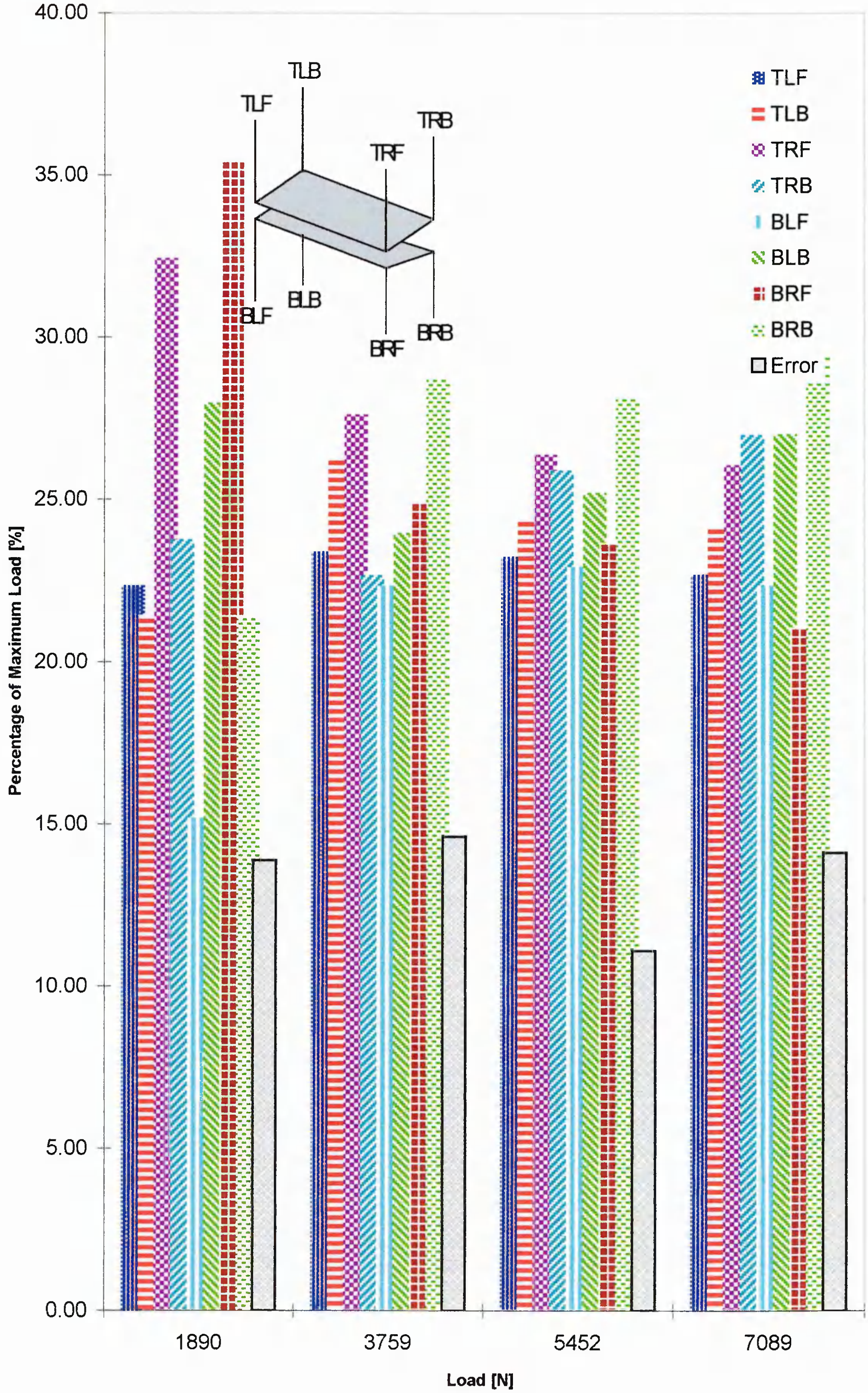



**Figure 2.4c Percentage of total load carried in the eight hanger rods. Model joint 3, slope across width, top plane parallel.**

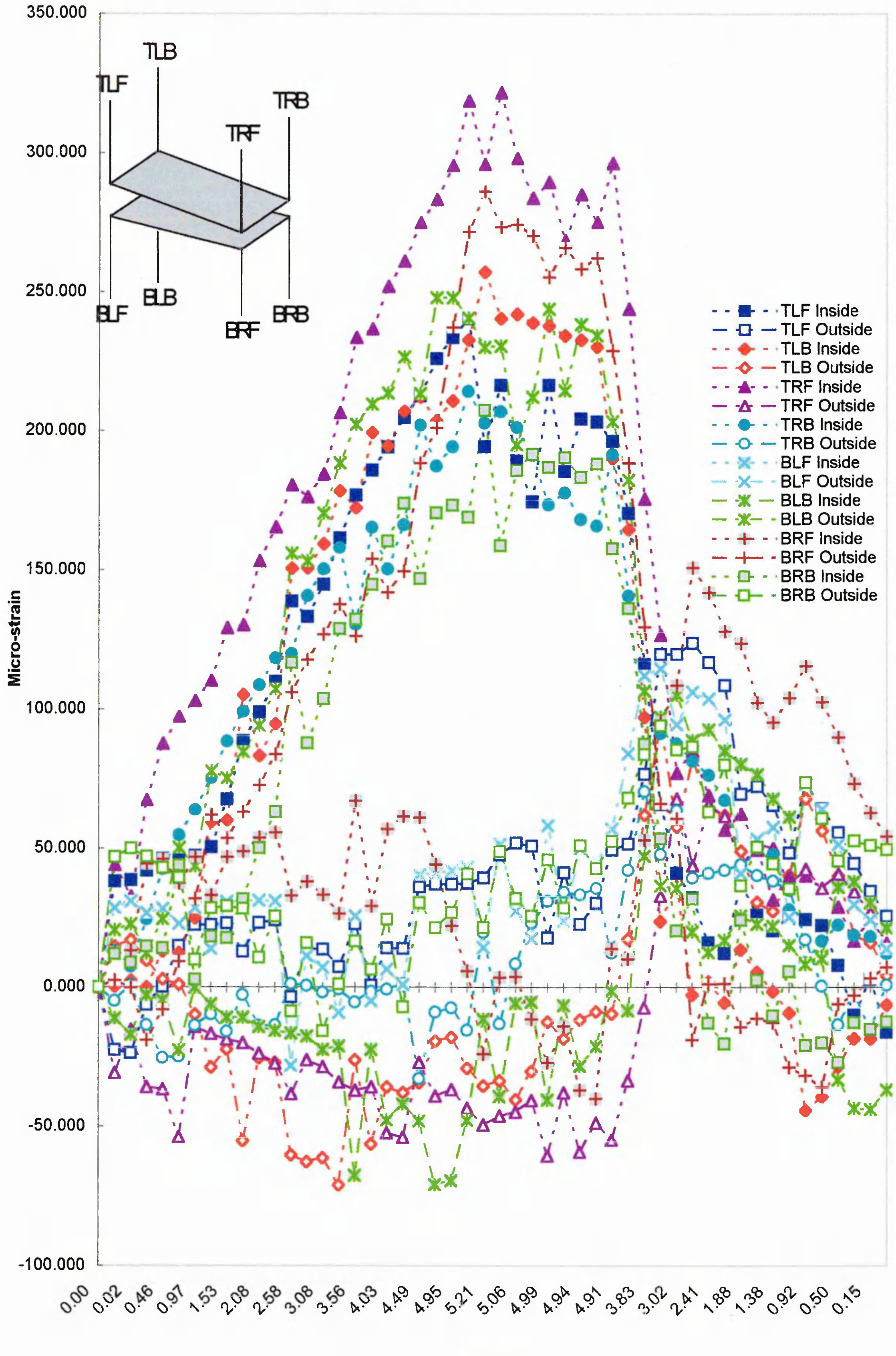



**Figure 2.5a: Distribution of bending strain in all eight hanger rods. Model joint 4, slope along length, top plane horizontal.**

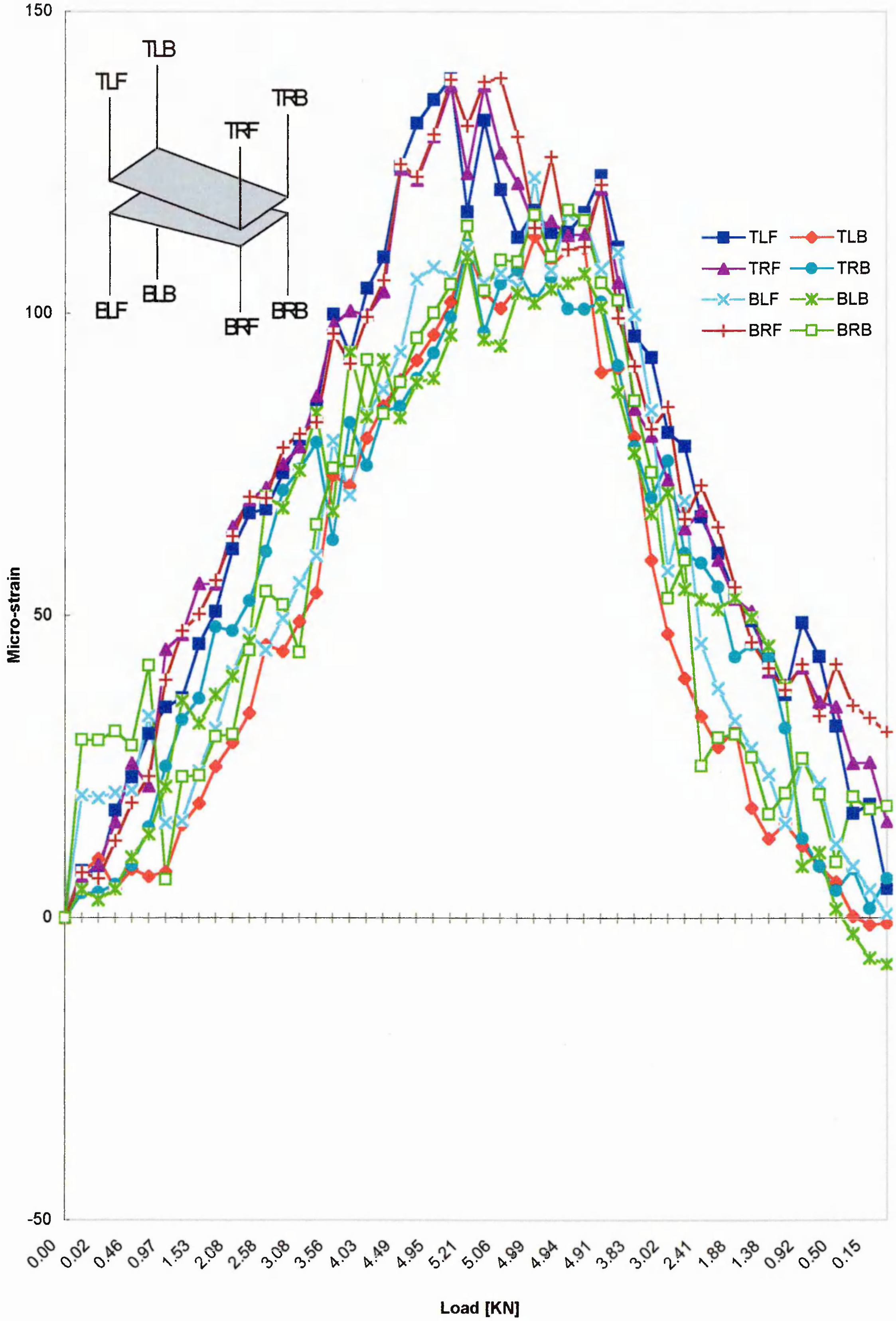



**Figure 2.5b: Average distribution of strain in all eight hanger rods. Model joint 4, slope along length, top plane horizontal.**

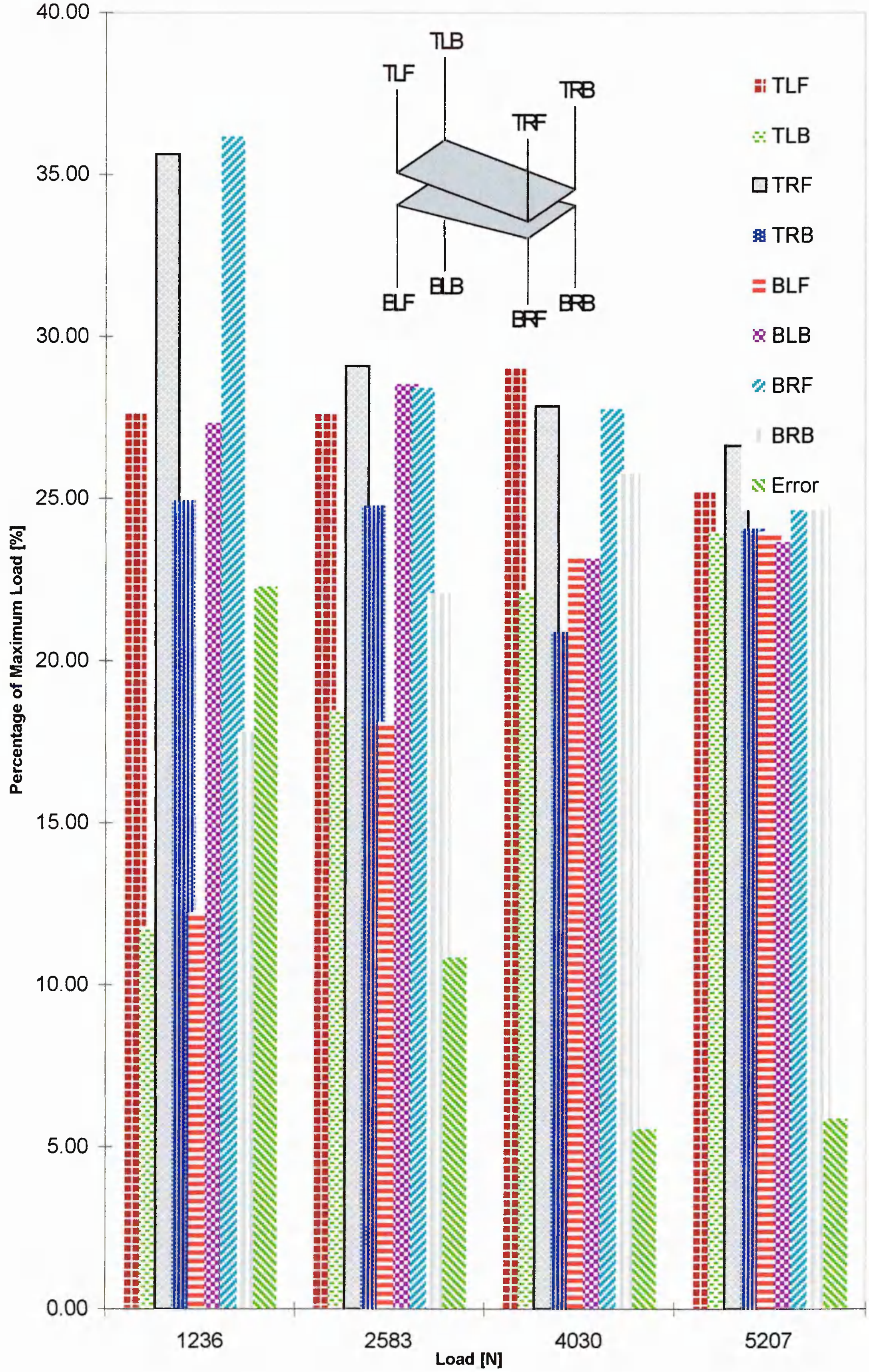



**Figure 2.5c: Percentage of total load carried in each of the eight hanger rods. Model joint 4, slope along length, top plane horizontal.**

## 2.5 Discussion of Experimental Results

### 2.5.1 Analysis of Strain Distribution

The appraisal of the tensile testing methodology must consider two aspects. Firstly, the axiality of the line of force and the nature of any eccentricities induced into the system by either the design of the test rig or the presence of a test sample. Secondly, consideration must be given to any eccentricity induced into the test sample via the testing apparatus or through flexing of the sample itself.

Figures 2.2-2.5 a) demonstrate that there is evidence of bending in the hanger rods; the outside of the rod is placed in tension and parts of the inside of the rod are in compression. For the parallel model joints, Figure 2.3a) and the joint with slope along length, Figure 2.5a) the magnitude of the compression strains are limited and tensile strains tend to dominate. However, for the deformed model joint with the slope across width, Figure 2.4a), there is evidence of considerable compressive strains induced in the bottom left back (BLB) and bottom right back (BRB) hanger rods.

This excessive bending is induced by the difference in joint thickness of 5mm across the relatively short length between hanger rods. The deformed model joint sloping along length does not depict the same exaggerated bending since the deformity of 5mm is tolerated over a longer span. In addition, the fixity for the top of the hanger bar in the plane normal to the length of the sample joint permits rotation which tolerates misalignment caused by joint deformity. The fixity in the plane normal to the width of the couplet does not permit the same degree of flexibility since the hanger rod passes through a 1mm clearance hole in the loading beam. If the angle of the loading beam is not perpendicular to the hanger rod, due to the angle of the spreader beam tolerating joint deformity across the bed width, this will cause fixity at the end of the hanger rod and will induce moment transfer from the loading beam to the hanger rod, as depicted in Figure 2.6.

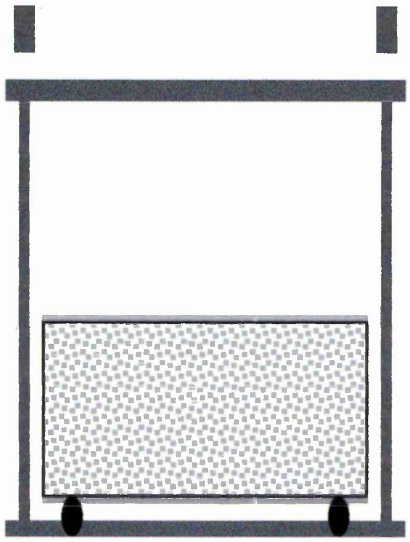


**Figure 2.6a: Schematic Representation of Hanger Cradle.**

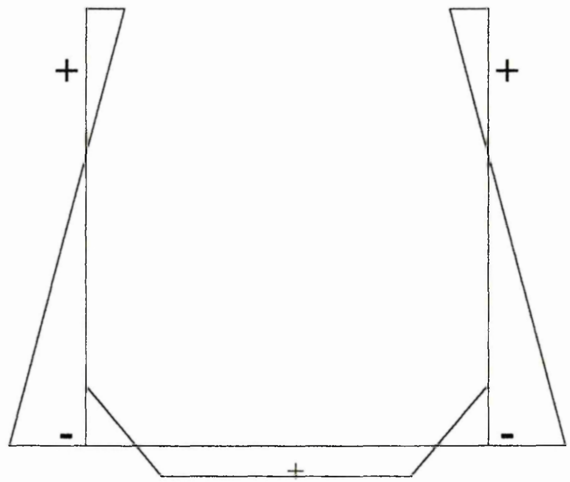



**Figure 2.6b: Theoretical Bending Moment Diagram for Hanger Cradle.**

The loading bar can be analysed as either a simply supported beam or an encastre beam but due to uncertainty of end fixity, an analysis somewhere between the two might be more appropriate. The hanger rods form the reactions with two equally concentrated loads induced by the couplet contacting the raised loading points. Bending is induced in the loading bar as force is applied to the test couplet. Although the test rig was designed to limit rigidity, the cup and cone connection between the hanger rod and the loading bar demonstrates a certain degree of fixity due to friction, which increases as load is applied.

As a result of this end fixity, the loading bar experiences a hogging moment at its end. Subsequently this moment is transmitted to the hanger rod which induces compressive strain to the inside of the rod along its length. Referring to Figure 2.6b, it can be demonstrated that the point at which compression in the hanger rod occurs, will be dependent upon the fixity at both ends of the hanger rod.

For the timber model joints, Figures 2.3 to 2.5, it can be seen that the curves are not smooth since there is a certain amount of bedding-in as the loading points compress the plywood.

The magnitude of strain in the outside of the rods, induced by bending, could be under represented if the strain gauge orientations are not perfectly located on opposite faces or the bar turned slightly so that the gauges were not in alignment with the loading beam. Such misalignment could mean that a lower value of strain was recorded and could explain why there is a difference in tensile and compressive strains in each of the bars.

Figures 2.2 to 2.5 b) show the average strain recorded by two gauges on each hanger rod. The curves represent the true tensile loading in each of the rods without the effects of bending. For the timber model joints in Figures 2.3 to 2.5 b), the strain profiles run parallel to each other demonstrating that the strain is carried equally in each of the bars. However Figure 2.2b for the test with no model joint shows that there is a severe discrepancy in the pattern of loading and that the majority of the load appears to be carried by diagonally opposite hanger bars. This suggests that the samples could span diagonally and not seat on all four points of contact. Generally, three support points would satisfy static equilibrium with the fourth point providing a degree of freedom. The disproportionate strain distribution for this test was surprising since this condition was intended to represent a perfect plane parallel system. However, upon consideration it is believed that the reason this test demonstrated such eccentricity is due to the short length of the test system. Because the test only utilises one set of hanger rods, the length of the test system is halved. The axiality of the test system relies upon the apparatus and couplet being allowed to hang with gravity before being levelled. However, such a short length exaggerates any misalignment because rigidity of the system is greater. It is considered therefore that the advantage of the adopted test apparatus lies in the overall length of the testing apparatus, which promotes flexibility. Other forms of testing, such as the ASTM crossed-couplet test apply very rigid apparatus which employ no tolerance of misalignment.

Figure 2.2 to 2.5c) show the distribution of load in the individual hanger rods for quartiles of the peak load. The general trend for all joint types is that the distribution of load becomes increasingly less varied as the load increases. This was to be anticipated since at the commencement of loading there is a certain amount of "bedding-in" as the loading points compress the plywood. For the first joint tested, for reasons explained above, there is a disproportionate spread of load and the error in the system marginally increases with load. For the timber model joints Shown in Figures 2.3 to 2.5c) the hanger rods each carry approximately a quarter of the total load in the test system. While there is some variation between the proportion of load carried by each hanger rod, there is no obvious pattern to the proportion of load distributed between corresponding opposite rods, as may have been anticipated for the model joints with in-built deformity. It is evident that variation becomes less pronounced as the load increases; the value of error, which represents the difference between the total load calculated from the strain in the hanger rods to the load recorded by the load cell, reduces as the load is applied.

The strain analysis demonstrates that the critical region of joint deformity, which may influence axiality of the test system, is joint deformity across the width of the sample. This may be induced if care is not taken when removing the joint depth roller bars, since there is no means of ensuring that the direction of pull is perfectly parallel to the line of the joint depth. Deformity along the length of the sample is less sensitive and this could be due to the freedom of movement of the hanger bars relative to the loading plate. When considering the degree of movement provided in the plane normal to the length of the couplet, the hanger bars are permitted to rotate on a simple bearing and therefore tolerate some joint deformity.

Fixity in the plane normal to the width of the couplet increases due to friction as load increases. If the loading bar is not perfectly perpendicular to the line of the hanger rod, the clearance for the hanger rod passing through the hole in the spreader bar is reduced, generating moment transfer.

The error or discrepancy between the theoretical and experimental load can be accounted for by variation in bar cross sectional area, orientation of the strain gauges on the rods and orientation of the rods themselves in relation to the test system. It was observed that the rods could turn in their seating as the load was applied which indicates that there may be torsion forces occurring in the bars.

With regard to transfer of forces to the sample joint, other than direct tension, this seems unlikely since the couplet is simply supported on the contact points. If on the other hand the original knife-edges had been employed, there would be more opportunity for transfer of horizontal load through friction along the knife-edge. The fundamental principal behind direct forms of testing would appear to be reliance upon minimal contact area between the couplet and the loading apparatus. The modification of the supports from knife-edge to four contact points is important, since it dramatically reduces the area of contact between the test specimen and the loading apparatus and also reduces the possibility of friction forces transferring to the test specimen in two directions as opposed to one. It can be argued that the inclusion of two contact points induce higher bending moment to the loading beam since the loading points are point loads positioned further away from the support of the hanger rods. The original purpose of the knife-edges was to provide uniformly distributed load over the entire width of the bed joint. In practice however, this was seen not to occur due to the deviations in flatness of the brick bed-face.

The only isolated unknown is the degree of flexing within the sample spanning between the loading beam along the sample length. Flexing could induce premature tearing of the interface bond, with edge effects reducing the fracture energy; this would not be indicative of true direct tension. An approximate assessment of the maximum deflection likely to occur in the lower brick was made, adopting a moment of inertia of $2.3 \times 10^6 mm^4$ for a standard format brick and a modulus of elasticity of 11 $kN/mm^2$[29]. This calculation showed that the deflection would be negligible at around 0.0004 mm for an ultimate load before failure of 2000 Newtons.

In conclusion therefore it has been determined that the test apparatus adopted is capable of measuring the direct tensile bond strength of brick/ mortar interfaces, provided that sample joint deformity is not too extreme, particularly across the width. The only recommendation materialising from this study is concerned with the fixity of the hanger rods and the loading bar. It is proposed that one possible modification of the test apparatus could be to replace the loading beam with a simply supported beam. This could be achieved by having two hanger rods in each corner of the bearing plate. A dowel connecting the two rods together could therefore be used to provide simple support to an independent loading bar. However, provided joint deformity is not too extreme, there seems little evidence that any bending induced into the hanger rods influences either the distribution of direct tension or transmits eccentric forces to the test sample. A further modification would be to dispense with the cup and cone type seating for the hanger bar screw nuts, since these were seen to turn during loading, inducing torsion into the bars. One possible alternative for this would be to introduce a washer arrangement and to reverse the screw nuts in order to present a flat bearing face.

## 2.6 Uncertainty in Measurement of Tensile Bond Strength

### 2.6.1 Experimental Errors in Evaluating Tensile Bond Strength

The final bond strength value recorded provides an indication as to the quality of the brick-mortar interface bond and allows for comparison of average bond strengths for different sample treatments. The final tensile failure strength recorded however is generated by the isolated measurement of several primary components, each of which contain their own individual errors. It is therefore important to consider the cumulative influence of these uncertainties before accepting the final bond strength value or using it to make comparison between other samples. Table 2.1 lists the possible sources of error likely to occur in the standard assessment of tensile bond strength of brick/ mortar interfaces.

**Table 2.1: Uncertainty in Assessing Tensile Bond Strength**

| **Description of Error** | **Accuracy** | **Bond Strength Uncertainty** | **Percentage Error** |
|---|---|---|---|
| Measurement of peak load from graph | ± 1mm | ± 0.00133 N/mm² | 1.0 % |
| Measurement of bond area (length x width) | ± 1mm | ± 0.0034 N/mm² | 2.55 % |
| Weight of lower brick (approx. 2000 g) | ± 19.6 N | ± 0.00131 N/mm² | 1.0 % |
| **Maximum Cumulative Uncertainty** | | **± 0.006 N/mm²** | **4.55 %** |

Values are based upon an average failure load of 2000 N and an assumed bond area of 15000 mm².

Accuracy of peak loads are based on measurements to the nearest millimetre from the output graph. Bond areas are measured across the centreline of the failure joint in both length and width directions to the nearest millimetre. These measurements may therefore differ from measurement along the edges of the joint. Measurement of bond areas are subjective since the failure surface is unlikely to be truly rectangular. It is considered that results are more consistent if an assumed bond area is used, based upon the nominal length of the mortar joint being 150mm and the nominal width of the joint being dependent upon an average width of the particular brick used, typically 102.5mm. A study of comparisons between measured and nominal bond area is recorded in section 2.6.2.

During testing, the weight of the lower brick upon the force applied to the joint is generally ignored since its influence is limited. However once the bond has failed, the datum on the plotter returns to a new level, with the discrepancy being equal to brick weight. Although the error can be considered negligible at $\pm$ 0.00131 $N/mm^2$, unless clarified, the presence of the lower brick can further complicate measurement of the ultimate peak load.

It can be concluded that primary measurements have little affect on final bond strength result. Error in the final result could be equal to a combination of maximum error in parameters used to calculate the result. The cumulative error of 5% of the total load is considered acceptable. Although conservative in its approach, this study combines errors in the most detrimental way to determine maximum possibly error likely to be present. The main contribution to uncertainty in tensile bond strength is generated by the measurement of bond areas and warrants a separate investigation of the validity of using a nominal bond area.

### 2.6.2 Study of Bond Areas

The bond areas of 60 failed couplets made from Fletton bricks were measured at the centre of the intact mortar bed, across the width and along the length to determine an average bond area for the population.

The average bond area was determined as $15047mm^2$ and the standard deviation was 229. The standard error was calculated as 29.86. Hence, a 95% confidence interval was calculated as $15047 \pm 59.72$ $mm^2$. This means that one can be 95% confident that the mean bond area for the population of bond areas contained within this thesis will lie within the above range.

Having determined that the chosen sample was representative of the population, the mean bond area of the sample was compared to a single theoretical value of $15000mm^2$ nominal bond area using a one sample t-test.

<u>It was determined that bond areas do not differ significantly from a nominal bond area of $15000mm^2$ ($t=1.58$, $d.f.=58$, $p>0.05$).</u>

This identifies that an assumed nominal bond area does not significantly differ from measured bond areas and because measurement of bond areas is subjective, adoption of a standard bond area will help to induce more consistency in comparative results.

### 2.6.3 Assessment of Workmanship

Although the couplets made in this programme of research were produced entirely by one trained operator, it was considered useful to investigate the potential variability in sample manufacture for a number of inexperienced operatives, using the couplet forming jig described in section 2.2.2. A comparison was made between the bond strengths at 7-days for couplets made by an experienced operator with five years experience of using the test rig, with construction students who had not used the apparatus before and most of whom were inexperienced in bricklaying. A two-sample t-test was used to examine the influence of workmanship upon bond strength. It was found that the influence of workmanship upon tensile bond strength of masonry couplets using the procedure outlined above was not significant at 95% confidence ($t=1.58$, $d.f.=31$, $p>0.05$).

### 2.6.4 Analysis of Variance in Bond Testing

When considering bond strength results it is useful to compare results from different treatment types. This will demonstrate both the magnitude of quantified bond strength values to be expected and the degree of consistency in the bond strength measurement between samples of the same type. In masonry, variation of data sets is often measured as a coefficient of variation (CoV), which is essentially an expression of variation of the sample as a percentage of the mean.

Table 2.2 lists bond strength values and their corresponding coefficients of variation for a range of sample types. Examination of the CoV's gives an indication of the consistency in testing.

The values for CoV's are in the range of 20-30%, which is lower than other forms of bond testing reported in the literature in section 2.1.1. It should be remembered that the values shown here are for samples that have not been selected and therefore embody variability in suction profiles of the units.

**Table 2.2: Analysis of Variance in Bond Testing**

| Treatment/ Mortar | Number Of Replicates | Unit Type | Age Tested [Days] | Average Bond Strength [N/mm²] | Standard Deviation | Coefficient of Variation [%] |
|---|---|---|---|---|---|---|
| 1.1.6 | 8 | Eng. | 14 | 0.154 | 0.03 | 22.0 |
| 1.8 | 24 | Fletton | 28 | 0.075 | 0.02 | 27.5 |
| 1.6 | 23 | Fletton | 28 | 0.092 | 0.02 | 16.5 |
| 1.1.6 | 23 | Fletton | 28 | 0.135 | 0.03 | 25.1 |
| 1.2.9 | 24 | Fletton | 28 | 0.12 | 0.03 | 24.0 |
| 1.3 | 23 | Fletton | 28 | 0.128 | 0.02 | 17.9 |

Importantly for the appraisal of the chosen tensile testing methodology, the work reported in the following chapters identifies that there is a distinguishable difference in bond strength values between treatments. The statistical analysis of average bond strength values demonstrate that the testing methodology is capable of identifying discrete differences between treatments.

### 2.6.5 Appraisal of Failure Plane

The value of tensile bond strength obtained for couplets refers to the strength of the particular interface which fails under loading; the bond strength of the second interface remains unqualified. It was observed during this research that almost all bond failures occurred at the lower interface. This could be due to one of two influences; the nature of the test apparatus induces a lower plane failure or, the bond formation process results in preferential bond strength development of the upper interface.

It is argued by Taylor-Firth and Taylor[25] that "the arrangement of the lateral bars which apply the force to the interface is such that the inevitable moments introduced, tend to preferentially induce failure along the lower of the interfaces in the test area". Due to the limitation of joint depth, the lower loading bars are offset to the outside of the joint. As a result of this eccentricity, the lever arm from the mortar edge to the point of load application is increased for the lower plane. Deflection of the masonry unit would therefore initiate greater stress at the joint periphery in the lower bonding interface.

Notwithstanding the above, it was observed during experimental work that certain sample treatments demonstrated an unusually high proportion of top plane failures, contrary to the above theory. For example engineering bricks and suction adjusted bricks showed a higher than average proportion of top plane failures.

As a result of these observations, an experiment was conducted to determine whether the failure mechanism was dictated by the test apparatus or induced by the bond formation process itself. A set of couplets were made using the procedure outlined in section 2.2.2 using non-suction adjusted Fletton bricks and 1:1:6 mortar in accordance with the procedure outlined in section 3.2.2. In total 20 replicates were tested, distributed over two identical mortar batches. Half the couplets were tested in the orientation in which they were produced and cured and the other half were inverted before testing. In order to be reasonably confident that any bond formation process within the couplets had reduced, the samples were cured for 19-weeks before testing. The results are shown in Table 2.3.

**Table 2.3: Appraisal of Bond Failure Plane**

| As Made | Inverted | Failure Plane | Bond Strength$[N/mm^2]$ |
|---|---|---|---|
| | | bottom | 0.114 |
| - | | bottom | 0.103 |
| - | | bottom | 0.156 |
| - | | bottom | 0.167 |
| - | | bottom | 0.045 |
| | - | top | 0.201 |
| | - | top | 0.172 |
| | - | top | 0.199 |
| | - | top | 0.126 |
| | - | top | 0.126 |
| - | | bottom | 0.135 |
| - | | bottom | 0.139 |
| - | | bottom | 0.121 |
| - | | bottom | 0.091 |
| - | | bottom | 0.128 |
| | - | top | 0.152 |
| | - | top | 0.089 |
| | - | top | 0.122 |
| | - | top | 0.187 |
| | - | top | 0.117 |

Failure plane refers to the orientation of the sample as tested, not as manufactured.

The results in Table 2.3 clearly demonstrate that the preferential failure plane is not induced specifically by the testing arrangement. The samples that were inverted when tested would have shown bottom plane failure if the test rig were inducing the failure mechanism. A two-sample t-test was performed to examine the differences between sample means.

There was no significant difference between the average bond strength of samples tested in the orientation in which they were made (0.123 $N/mm^2$) and inverted samples (0.143 $N/mm^2$), ($t = -1.22$, $d.f. = 18$, $p > 0.05$).

If results had identified a significant difference in bond strength, this would have indicated that there was a preferential failure induced by the test rig. However no such phenomena occurred and therefore it can be concluded that the preferential failure mechanism is initiated during bond formation process either at the laying stage or subsequently during curing. Any preferential failure induced by the difference in lever arm between the upper and lower cradles was shown not to be significant.

## 2.7 Concluding Remarks

For investigative work involving the comparison of different treatment types and their influence on bond strength performance, careful consideration of the testing methodology must be given.

It has been demonstrated in the literature that flexural forms of testing rely heavily upon joint edge effects for evaluation of overall bond strength. Joint edge influences have important implications because flexural bond strengths may be overestimated and not representative of the overall bonding area. Furthermore, edge effects such as shrinkage, hydration or carbonation may be a function of the particular variable under investigation.

Bond strength values attained through direct tensile measurement also rely on an assumed bond area. However, bond area is a function of joint width across the bedface while section modulus is a function of the square of the width. In consequence, any under representation of width due to shrinkage cracks at the joint periphery are compounded by flexural calculations.

Flexural bending is both a function of the tensile and compressive strength of the mortar and tensile strength of the bond. The former bears no relationship to the later and therefore investigations of bond strength criteria should employ a test method which is not dependent on the structural behaviour properties of the mortar.

When conducting investigations to determine the nature of formation of the interface bond, it is considered that quantification of the bond strength alone is not sufficient to identify parameters leading to bond development. Indeed, the preferential failure of the lower interface may help to indicate the mechanisms at work in bond development.

This chapter has demonstrated that the Sheffield Hallam University test is capable of measuring direct tensile bond strength of brick-mortar interfaces. Interrogations of the test rig have identified that there is no evidence that the test imparts eccentricity to the test specimen, provided reasonable care is taken during sample manufacture.

The most important finding of this investigation is that different treatment types within the brick or mortar can be distinguished against a background of inherent variability associated with masonry. Other forms of testing which do not facilitate high sample representation cannot apply statistical tests with the same level of confidence.

It is therefore concluded that the two forms of testing, whether flexural or direct, both have useful functions. However, it is recommended that the two approaches are not compatible and should be used for different purposes. Flexural testing is considered a suitable representation of in-situ performance. Direct testing is suited to laboratory-based investigations and comparative studies.

Indeed, it is presented that the adopted method of testing, has in the past led many workers to consider a relationship between the compressive properties of the mortar and the tensile bond strength. It is demonstrated in Chapter 3 that there are parameters which benefit both, but that there is no specific relationship between them.

**References**


16. British Standards Institution. Code of Practice for the Use of Masonry: Part 1: Structural Use of Un-reinforced Masonry. BS5628: Part1: 1992.

18. ISO/DIS 9652-4. Masonry–Part 4: Test methods. British Standards Institution, London 1993.

19. CEN prEN1052-2. Methods of Test for Masonry. Part 2: Determination of flexural strength. British Standards Institution, London 1993.

20. SAA Masonry Code AS3700-1988. Masonry in Buildings. Standards Association of Australia, Sydney N.S.W., 1988.

21. ASTM E518/80. Flexural Bond Strength of Masonry. American Society for Testing Materials. 1981.

22. De Vekey, R. C. Testing Bond Strength of Masonry: The Brench. BRE Digest 360, April 1991.

23. C1072-86. Standard Method for Measurement of Masonry Flexural Bond Strength. American Society for Testing and Materials. December 1986.

24. Standard Test Method for Bond Strength of Mortar to Masonry Units. ASTM Designation C 952-91, 1986.

25. Taylor-Firth, A. Taylor, I.F. A Bond Tensile Strength Test for Use In Assessing the Compatability of Brick-Mortar Interfaces. Construction and Building Materials. Vol. 4, No. 2, June 1990.

26. De Vekey, R.C., Page, A.W., Hedstrom E.G. The Variability of Bond Wrench Measurements In UK, Australia and USA. BRE PD No. 265/93, 1993.

27. Anderson, C. Some Observations on Masonry Wallette Testing. International Journal of Masonry Construction, 2, (4), 1982.

28. Lovegrove, R., De Vekey, R.C. The Effects of Specimen Format on Flexural Strength of Wallettes. Proceedings of British Masonry Society.1, 1986

29. Lovegrove, R. The Effect of Thickness and Bond Pattern on the Lateral Strength of Brickwork. Proceedings of 1st. British Masonry Society Conference, London, 1986, abstract 41.

30. Drysdale, R.G., Hamid, A.A. Tensile Strength of Brick Masonry. Journal of Masonry Construction, No. 4 pp172-178, 1982.

31. De Vekey, R.C. The Influence of Mortar Type on the Flexural Strength of Masonry. Proceedings of the 7th International Brick Masonry Conference, Melbourne, Australia, Feb. 1985.

32. Drysdale, R.G., Gazzola, E. Influence of Mortar Properties on the Tensile Bond Strength of Brick Masonry. Proceedings of the 7th International Brick Masonry Conference. Melbourne, Australia, Feb. 1985.

33. Adams, M.A. Hobbs, D.W. Bond Strength of Brickwork from Couplet and Wallette Tests: Some Comparative Results. Masonry International Vol. 8, No 1, 1994.

34. Baker, L.R. Some Factors Affecting the Bond Strength of Brickwork. 5th International Brick Masonry Conference, Session 2, paper 9,1979.

35. Isberner, A.W. Properties of Masonry Cement Mortars. Designing Engineering and Constructing with Masonry Products. Gulf Publishing Company, Houston, Texas 1969.

36. Anderson, C., Held, L.C. The Effect of Sand Grading on Mortar Properties and the Tensile Bond of Brickwork Specimens. 6th International Symposium on Load Bearing Brickwork. British Ceramic Society. 1983.

37. Pearson, J.C. Measurement of Bond Between Brick and Mortar. Proceedings ASTM 43, 857, 1943.

38. Pluijm, R.V.D. Numerical Evaluation of Bond Tests on Masonry. Masonry International, Vol 9, No.1, 1995.

39. Palmer, L.A., Parsons, D.A. A Study of the Properties of Mortars and Bricks and Their Relation To Bond. National Bureau of Standards Journal, Rp 683. 1934.

40. McBurney, J.W., Copeland, M.A., Brink, R.C. Permeability of Brick-Mortar Assemblages. Proceedings ASTM. 46,1333,1946.

41. Thornton, J.C., Relation Between Bond and the Surface Physics of Masonry Units. Journal American Ceramic Society. 36, (4), 105, 1953.

42. Kampf, L. Factors Affecting the Bond of Mortar to Brick. ASTM. Special Technical Publication No 320 p67; Symposium on Masonry Testing, 1963.

43. Ritchie, T., Plewes, W.G. Moisture Penetration of Brick Masonry Panels. ASTM Bulletin (249), 39, 1960.

44. Hendry, A.W. Workmanship Factors and the Strength of Masonry. BDA Technical Note Vol. 1 No.6 pp 48-50.

45. Sise, A., Shrive N.G., Jessop E.L. Flexural Bond Strength of Masonry Stack Prisms. Proceedings of the British Masonry Society April 1988 No 2 pp 103-107.

# BOND STRENGTH PERFORMANCE CHARACTERISTICS OF GENERIC MORTAR MIXES

## Chapter Summary

*Chapter 3 investigates the bond strength performance of generic mortar mixes. By proportioning cement and lime to a constant mass of sand, using traditional volume ratios, it is possible to determine the influence of increasing cement content upon bond strength of mortars. Correspondingly, the addition of lime can be monitored as an independent additive. The presence of a constant mass of sand from a consistent source, across the array of chosen test mortar mixes, means that any change in water content between mix ratios, required to produce a given consistency, can be attributed to variation in the quantity of cementitious binder material and not to characteristics of the sand.*

# 3.0 Introduction

Workability is the term given to the measure of acceptability of mortar in its wet state as a bedding material for masonry units. It is perhaps the single most important consideration when assessing the properties of a mortar on site and is often dependent upon the control of the bricklayer. The continued strength and durability performance of the mortar, which is specified at the design stage, may be impaired by alteration of the wet properties of mortar, in order to achieve desired workability on site.

Workability performance of the mortar is an important commodity within the global practice of brickwork, since it not only promotes good workmanship and future durability of the brickwork but also helps optimise bricklaying efficiency.

In the laboratory, the mason's subjective assessment of the suitability of the mortar for laying masonry units is simulated by a set of procedures outlined in BS4551[46], which uses consistency, flow and water retention measurement to characterise mortar properties.

Mortar requires an exceptionally high water content to provide desirable workability. Pyle[47] estimates that of the water required to achieve the desired workability performance of a mortar, only 25 to 30 % is actually needed to sustain cement hydration. Bessey[48] argues that the compressive strength of cement:lime:sand mortars, as with concrete, is related to water-cement ratio; however for a given material and mix proportion, the water content for mortar is fixed according to consistence and workability requirements of the mason. Practically, it is the cement content alone which controls strength with any given sand. The addition of water for workability remains in the control of the site operative. The excess mixing water, if not removed from the mortar, will increase the water-cement ratio, resulting in a reduction in strength and durability performance of the mortar.

The extent of water removal from the mortar is a function of brick absorption capacity and suction force. It follows that the quantity of water removed will vary depending upon the unit suction characteristics and the resistance to water loss (retentivity) of the particular mortar.

If too much water is removed from the mortar bed, hydration of the cement will be inhibited. If insufficient water is removed, the remaining chemically unbound water will evaporate slowly, after the initial setting period, causing drying shrinkage and increased porosity of the mortar matrix.

An acceptable mortar therefore requires a water content which is elevated enough to lubricate the constituent particles to give satisfactory workability on the spot-board. Any excess water should be free to compensate brick suction demands. Compatibility between mix water content and unit suction characteristics is essential to achieve a durable mortar in the hardened state. It is a function of the constituent materials of the mortar to balance workability requirements against in service performance requirements.

The optimum water content can only be realised if there is sufficient particle surface area within the mix to absorb and retain water required for hydration purposes. It follows that the higher the proportion of hydrophilic materials such as cement and lime in the mix, the greater the surface area available for water retention.

Mortar mix design must therefore optimise packing and present sufficient constituent surface area for water adsorption. Traditional volume based mixes used volume batching for convenience and the proportion of cement or cement and lime, equal to approximately one-third the volume of sand, was considered to fill the voids between the sand particles. This concept forms the basis of traditional volume proportions defined in BS4551[46] and presented in Table 3.1 below.

**Table 3.1: Traditional Volume Mix Ratios Outlined in BS 4551**

| Designation | Cement | Lime | Sand |
|---|---|---|---|
| i | 1 | 0 to ¼ | 3 |
| ii | 1 | ½ | 4 to 4½ |
| iii | 1 | 1 | 5 to 6 |
| iv | 1 | 2 | 8 to 9 |

The procedure for batching laboratory mortar mixes for masonry purposes is outlined in BS 4551 and is designed to yield 15 kg total dry mass of constituents, which is based upon the capacity of a standard mixing bowl. In order to generate an equal mass of constituents for each of the mix designations, the sand quantity, cement, lime and subsequently water contents must vary across the mix array. The variation in each of the constituent materials is demonstrated in Figure 3.1a, below.

Comparable studies between mortar mix designations, using BS 4551 batching proportions are unable to distinguish the effect of change of one particular constituent in isolation, since sand, cement and lime proportions all change simultaneously.

This work promotes the use of a constant mass of sand for comparative investigations and concentrates upon maintaining the source and quantity of sand as a constant, while proportioning cement and lime constituents to yield a mix of equivalent volume ratio.

Consequently, the effect of change of any one variable can be monitored. This principle is demonstrated by Figure 3.1b.

The presence of lime within the mix can be monitored as an additive and its influence both on water content to achieve a given consistency and strength performance can be compared to non-lime mixes with the same quantity of sand and cement.

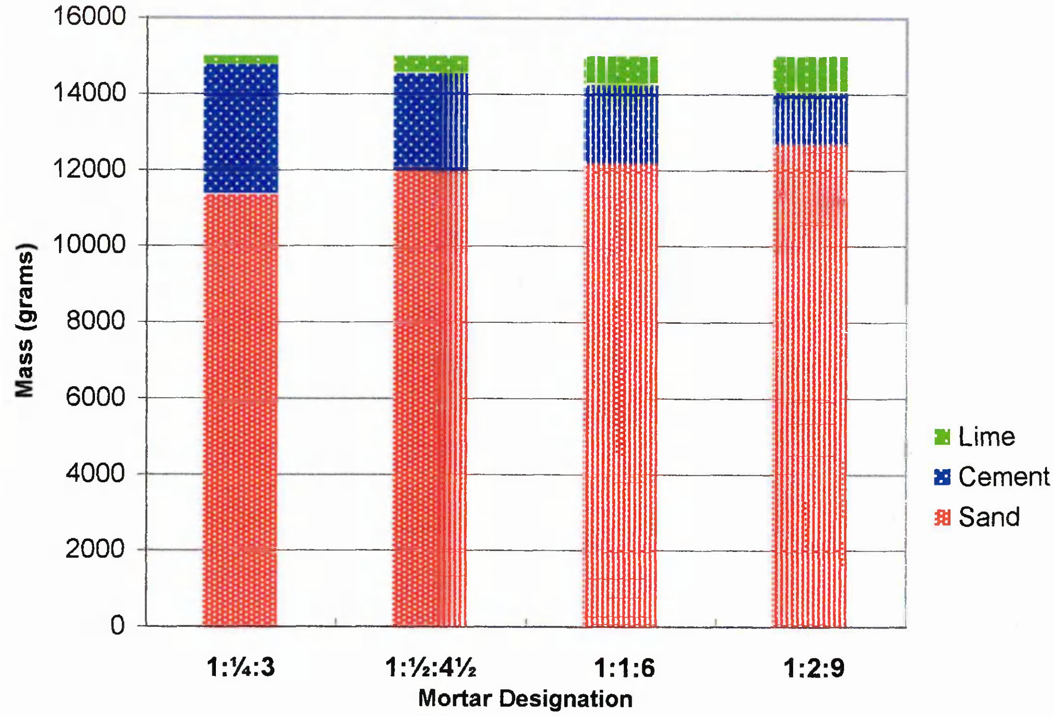



**Figure 3.1a: BS 4551 Laboratory Based Mix**
(Mortar mass yield constant at 15kg)

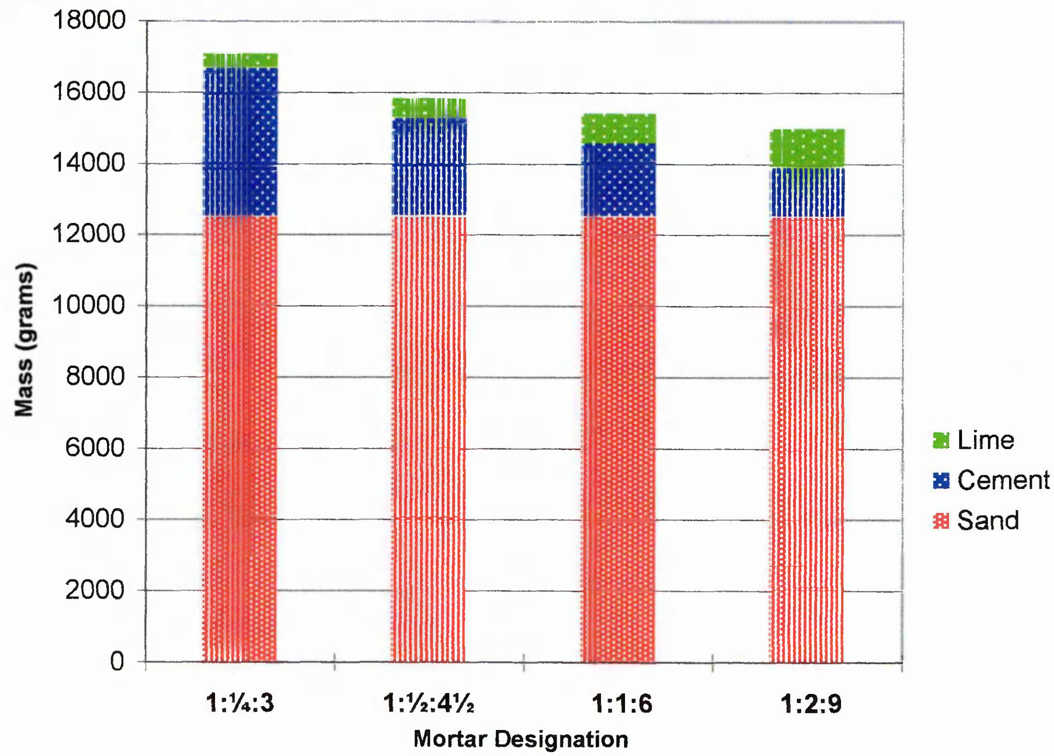



**Figure 3.1b: Constant Mass Sand**
(Mortar mass yield variable)

## 3.1 Discussion of Literature Relating to Mortar Properties

Until about thirty years ago the choice of mortars in the UK was limited to lime:sand or cement:lime:sand mixes. The introduction of admixtures and factory mixed mortars, including retarded mortars and masonry cements resulted in a greatly extended product range. Harrison and Bowler[17] estimated that in 1989 there were over 137 different mortar mixes available for specification. Despite advanced technologies for masonry cement and ready-mix mortars, all mortars tend to employ the same principles in their mix design in order to achieve a balance between workability parameters and a strong and durable mortar in the final state.

Concentration on generic mortar mixes is important since historically, bricklayers have made mortars from local materials, producing what their skill taught them to be the most suitable properties for the job. This practice is still common place on many construction sites, despite the advance in ready-mixed mortars.

Taking into account inherent variability in site mixing practice, together with variation in sand type, there exists an almost limitless array of mix specification available. The focus of this research remains with generic cement:sand and cement:lime:sand mortar mixes. It is imperative that an understanding of the behaviour and performance of generic mortars is reached, to enable the effective employment and development of more sophisticated mortars.

Sand quality is the principle contributor to mortar variation since sand forms the major constituent in any mix design, contributing to around 80% of the dry ingredients by weight. The fact that many traditional building sands do not conform to the limits set out in BS 1200[49] has led to an interest in the study of the effect of grading and other characteristics of sand upon mortar strength and in particularly bond strength.

The work reported in this chapter does not attempt to further the study of the influence of sand type upon bond strength performance, but rather aims to limit variability of the sand by keeping the quantity and constitution constant across the array of mix variations. The effects of changing levels of other constituents, such as cement and lime, can therefore be observed. Notwithstanding, it remains important to consider the influence that sand properties may have upon mix workability, retentivity and ultimately bond strength performance and these are discussed within the following literature review.

Bessey in 1966[48] identified average particle size, particle size distribution, shape and in particular impurities such as clay content, as the principal factors influencing mortar performance, such as workability, water retentivity, strength and durability.

The literature available regarding the influence of cement and lime levels upon tensile bond strength of mortar is limited. Perhaps one explanation for this is that traditional batching methods, such as those promoted by BS 4551, do not allow for the isolation of additives, as discussed previously. A further explanation could be for the reasons discussed in the literature review to Chapter 2; the apparatus available for comparative work on bond strength tends to reinforce the general assumption that bond strength is related to compressive and tensile properties of the mortar, which are controlled by cement content alone.

### 3.1.1 Workability

Bowler, Jackson and Monk[50] summarise the masons assessment of mortar workability as its ability to hang on the trowel, spread easily to form an even mortar bed, adhere to vertical surfaces and allow placement of units without squeezing out of joints.

RILEM[51] considers consistence and plasticity as the main factors constituting the property of workability. Consistence has been defined as that property of mortar which resists deformation, while plasticity is seen as the ability of mortar to retain its deformed state. The RILEM definition of plasticity incorporates the characteristic of thixotropy (shear thickening) which, in mortars is seen as the ability to hang on the trowel as a cohesive mass, to flow freely from the trowel and then develop relatively quickly a rigid structure when placed in contact with the masonry unit.

McIntosh[52] states that the working characteristics of the mortar have two essential properties; consistence which is affected by the amount of water added and cohesiveness which is improved by the presence of fine material including cement, lime and air. In addition the presence of silt and clay within the sand has the effect of making the mix more workable and provides a property known to the mason as "fattiness".

Aside from hydrating the cement, the other function of the mix water is to lubricate the binder and sand particles in order to impart workability. This lubricating water subsequently evaporates, leaving voids within the mortar matrix and may give rise to excessive shrinkage. The two main characteristics of the aggregate which influence the water required to lubricate the particles are total surface area and particle interference. Since the larger the surface area of the mix, the more water required to lubricate the surface of the particles. McIntosh recognises both these as being a function of grading. A large surface area can be a result of a fine grading or the presence of a large proportion of sharp, angular particles. Very fine particles can demonstrate their own lubricating mechanism and therefore less water is required to impart workability.

### 3.1.2 Water Retentivity

Retentivity of the mortar is described as its ability to retain water and maintain workability, even when in contact with an absorptive brick. Watstein and Seese[53] observed that for a mix of given properties the water retentivity of cement:lime:sand mortar was dependent primarily on the plasticity of the lime used. Palmer and Parsons[39] examined the effect of water retentivity on bond by using permeability tests on small brickwork panels. They determined that wallettes constructed with porous bricks laid dry, were more watertight with mortars of high retentivity than with mortars of low retentivity. Hogberg[54] however found that mortar with poor retentivity gave good bond to absorbent bricks. This phenomena could be explained by the fact that initial suction rate, which measures the quantity of water removal by the brick in the initial 60-seconds, is not sufficiently sensitive enough to measure the suction force or the total quantity of water removed by the brick. Chapters 6 and 7, examine more closely the potential mechanisms of brick water absorption. Previous workers could possibly have used bricks with high water absorption, in anticipation of high water removal from the mortar, but which may not hold properties of high suction force which is the true measure of water retentivity of the mortar.

Water retentivity can be increased by the use of air-entraining agents or finely ground particles. The presence of clay has also been shown to influence retentivity. Currie and Sinha[55] observed that well graded sands reduce void content and prevent separation of materials in the mortar mix, which in-turn reduces bleeding.

### 3.1.3 Grading and Packing

The majority of sands used in the UK are derived either from natural deposits such as glacial or marine sands, or are produced from crushed rock. Often the coarse fraction exceeding 5mm are screened off and sands are rarely washed or re-graded in any way. Therefore, sands often have wide range of grading and average particle size.

Before 1984, there was a general grading and a special coarser grading for engineered brickwork. After 1984, the standard sieving method was changed from dry to wet sieving which increased the measured amount of fines. Subsequently, the old general grade was promoted to being the structural (S) grade and a new general (G) grade with wider limits was introduced.

Work by Currie and Sinha[55] indicated that finer sands often did not attain the mortar cube strengths specified in BS 5628[16]. In addition, work by Kloppers et al[56] showed that flexural bond strengths were inferior for finer sands.

Silica has a specific gravity of 2.635. Dry compacted natural sands generally have bulk densities of between 1400 and 1800 kg/m$^3$. Therefore, the void content lies between

32% and 47%. As discussed previously, mortars traditionally have been volume batched using 1:3 binder to sand ratio, based upon the belief that the average void content for sand lies around 33%. Beningfield[57] argues that this ratio could in fact vary from about 1:4 for fine sand to 1:2.5 or less with coarser sands.

Lee[58] determined that water requirement of mortar for a standard consistence increased with the void content of the sand used. The water contents also seemed to vary with the fineness modulus of the sand. Fineness modulus is defined as the cumulative percentages retained on the sieves of the series 150, 300, 600μm, 1.18, 2.36, and 5.00mm. The coarser the sand, the higher the value of fineness modulus. Lee measured the specific surface of the sand by nitrogen adsorption, but found it to have no relationship with the water content of the mortar. Lee acknowledges that the nitrogen adsorption method used gives both the internal and external surfaces of the particles and therefore it is possible that the specific surface has been exaggerated and that only a small proportion of the total surface area of the aggregate, the external surface, has any influence on the water requirement of the mortar. The water requirement may also be affected by surface tension forces preventing the water from penetrating the smaller pores and fissures.

### 3.1.4 Mortar Strength and Durability

Compressive strength testing of mortars is perhaps not sensitive enough to detect influences of sand characteristics alone and mortar strength is dictated principally by the quantity of cement. In order to protect from frost damage, a limit on cement content is generally imposed, however with well-graded sand lower cement contents can be used. According to Beningfield[57] this is because the improved packing of the sand particles permits the cement to fill the voids.

Bowler[59] points out that sand particle size and grading influences the water cement ratio and pore size distribution of the mortar, which in turn may have profound influence upon susceptibility of mortar to chemical and frost attack. Finer graded sands give high water cement ratios and hence greater permeability of the mortar matrix.

Harrison[60] demonstrated that the more finely graded sands were less durable in frost. The richer the cement content of the mix, the more durable. However Harrison found that brick suction background had a greater affect on durability than either sand grading or cement content. Harrison also pointed out that finer sands show higher levels of carbonation.

Lee[58] determined that the lowest consistence and therefore water cement ratio did not always produce the highest mortar strength.

### 3.1.5 Shrinkage

It is generally accepted that finer grained sands require more water to attain reasonable workability and this causes reduction in strength and increased drying shrinkage. Lee[58] identified that a poorly graded sand, having low dry compacted bulk density, would require a high proportion of material to fill the voids and suggested that if this material were to be cement or water, then high drying shrinkage would result. Poorly graded sand can be improved by blending with other sands or by adjusting the lime content or adding an air-entraining agent so that discrete air pockets are formed.

Harrison[60] suggested that high mortar porosity and drying shrinkage tended to be greater with finely graded sands due to the high water to cement ratio.

Bessey[48] states that high lime, low cement mixes, have lower drying shrinkage. Shrinkage due to carbonation is important but is considered to occur from the surface, moving progressively into the joint, over longer periods and as a result will not set up the same stresses as drying shrinkage within the mortar bed.

Davison[61] determined no consistent change in shrinkage with increasing air content of the mortar. Lowest shrinkage occurred in mortars with the highest lime content. A 2% reduction in compressive strength with 1% increase in air content was observed.

### 3.1.6 Tensile Bond Strength

Held and Anderson[62] found that the higher the fines content the lower the bond strength. However water cement ratios also fall with increasing fines content, perhaps due to fines aiding workability while not adsorbing water. Held and Anderson also witnessed a reduction in air content as fines increased, presumably due to optimum packing within the mix. They found a marked increase in density with increasing fines. Their results showed that mortar compressive strength was not a reliable indicator of tensile bond strength. It was determined that a marked decrease in bond strength with increasing fines content occurred and this was particularly pronounced for wallette flexural strengths. The mortar compressive and tensile strength increased with the percentage of fines in sand while bond strength decreased. The authors also found a negative linear relationship between wallette strength and specific surface area of the sand.

Drysdale and Gazzola[63] found that flexural strengths of 7 brick high piers tested with a wrench type device showed that flexural bond strengths were higher for 120% flow than for 110% flow at 95% confidence levels. Use of lime provided a significantly increased flexural bond strength than masonry cement mortars.

In a further investigation Hogberg[64] examined the effect between the ratio of binder to sand and determined that with absorbent bricks the bond strength was usually improved when the amount of sand in the mortar was increased. Since the binder-sand ratio and the water-binder ratio are closely interrelated, increase in sand content results in a higher water-binder ratio. For example, a 1:3 and 1:6 cement-sand mortar will require roughly the same amount of water to provide a workable mix. Hence the water-cement ratio of the 1:6 mortar will be twice as high as the 1:3 mortar. Hogberg believes that there are mortars which can be used regardless of the suction of the base material and points the way to an optimum mortar mix which will not be sensitive to unit suction background.

De Vekey[31] found lime mortars to give the best flexural strengths using wallettes, while plasticised mortars gave the lowest overall performance in flexural tests.

Masonry cements are specially blended for use in mortars and consist of Portland cement and finely divided filler to provide the amount of fine powder necessary to achieve plasticity, whilst avoiding excessive strengths produced by elevated cement contents. Masonry cements also include air entrainer to impart plasticity; they are most commonly used with sand in the ratio of 1:5.

Plasticisers are generally air entrained surface active agents which work by entraining minute air bubbles on the surface of the aggregate and thus provide a lubricating affect, allowing workability to be achieved at lower water-binder ratios and lower binder-sand ratios. A normal mortar will have between 3% and 5% air by volume compared to an air entrained mortar, which may contain 10 to 15%.

Retarded mortars clearly have economical advantages over site mixed mortars, however Bessey[48] suggests that the retardation process, which can indirectly be affected by various additives, is not yet properly understood. If retardation is not in balance with brick background suction, the mortar may never set or alternatively, may stiffen too readily.

### 3.1.7 Summary of Literature

Evidence embodied in the review of literature regarding mortar properties points strongly to the influence of sand particle size, grading and hence surface area having the most marked influence upon water requirements to aid mix workability.

Sand forms a high percentage of mortar constituents, typically about 80% by mass and therefore any variation in sand sources will influence mortar properties. In addition variation in sand quantity between different mix specifications using BS 4551 batching procedures will also influence mortar properties and performance. As a result, changing cement and lime contents across mix designations are observed against a background of different sand quantities.

Using a constant source and mass of sand enables the influence of cement and lime additions to be identified in isolation.

Cementitious content, while controlling compressive and tensile strengths of the mortar and porosity, have little influence upon bond strength development and in cases of high shrinkage, may even be detrimental to bond formation. Lime on the other hand would appear to have advantageous properties, however previous research remains unable to identify whether these effects are due to subsequent reduction in cement content in order to maintain a given binder-sand ratio, or due to the water retaining properties of the lime. Again using a constant mass of sand batching procedure, the influence of lime can be investigated as an additive.

Research suggests that the actual input of constituent proportions upon mortar performance is due to the initial workability properties imparted by the percentage of fine material available. Continuing strength development and durability may be further enhanced by cement content or addition of air entrainer but the effect of these is not as marked as the influence brick background suction has on mortar properties.

# 3.2 Methodology of Constant Mass Sand Investigations

### 3.2.1 Constant Mass Sand Philosophy

The array of mixes in Table 3.2 below were chosen to encompass the traditional volume proportions outlined in BS 4551. The addition of lime is made to maintain the ratio of one part cementitious component to three parts sand, as the cement content reduces. In order to determine the effect of change in cement content with changing lime content and the subsequent affect this has upon water content for a given consistency, it is necessary to isolate each variable in turn. The result of changing mix proportions can then be investigated in relation to bond strength performance.

Equation 3.1 calculates the quantity by mass (M) of cement and lime required to produce a mix equivalent to traditional volume (V) mixing, but based upon a selected mass of sand. The variation in bulk density (BD) for different sands can be accommodated within the formulae.

$$M_{cement} = \frac{V_{cement}}{V_{sand}} \times \frac{BD_{cement}}{BD_{sand}} \times M_{sand\,(\,selected\,)} \qquad \text{Equation 3.1a}$$

$$M_{lime} = \frac{V_{lime}}{V_{sand}} \times \frac{BD_{lime}}{BD_{sand}} \times M_{sand\,(\,selected\,)} \qquad \text{Equation 3.1b}$$

It has been found that 12.5 kg of sand (irrespective of mix proportions) will not dramatically change the mixing capacities associated with the standard mixing procedure which yields a 15 kg mix.

Table 3.2 below demonstrates the application of the formulae for the array of mixes adopted in this experimental program. The values are shown graphically in Figure 3.2, which depicts the mass of cement and lime added for each chosen bulk density of sand, against the volume ratios for cement and sand. Sand bulk densities are based upon 1450, 1675 and 1900 kg/m $^3$ as outlined in BS 4551[46], section 7.3.3. Bulk density for Portland cement and hydrated lime are taken to be constant at 1450 kg/m$^3$ and 575 kg/m$^3$ respectively.

**Table 3.2: Traditional Volume Proportioning to BS 4551.**

| **BS4551 volume ratio [c:s] [c:l:s]** | | **Percentage by mass of total dry mass of mix** | | **Mass of constituents for a laboratory based 15 kg mix [kg]** | | | |
|---|---|---|---|---|---|---|---|
| | **Designation** | **cement:sand** | **cement:lime:sand** | **cement** | **lime** | **sand** | **total** |
| **1.3** | i | 22.8:77.2 | _ | 3.42 | _ | 11.58 | 15 |
| **1. 4½** | ii | 20.5:79.5 | _ | 3.075 | _ | 11.93 | 15 |
| **1.6** | iii | 14.0:86.0 | _ | 2.1 | _ | 12.9 | 15 |
| **1.8** | iv | 10.5:89.5 | _ | 1.575 | _ | 13.42 | 15 |
| **1.¼.3** | i | _ | 22.9:1.5:75.6 | 3.44 | 0.225 | 11.3 | 15 |
| **1.½.4½** | ii | _ | 17.0:3.1:79.9 | 2.55 | 0.465 | 11.99 | 15 |
| **1.1.6** | iii | _ | 13.6:5.1:81.3 | 2.04 | 0.765 | 12.2 | 15 |
| **1.2.9** | iv | _ | 9.0:6.4:84.6 | 1.35 | 0.96 | 12.69 | 15 |

**Table 3.3: Batching Proportions for a 12.5 Kg Constant Mass of Sand Mix.**

| Volume ratio [c:s] [c:l:s] | V ct / V sand | V lime / V sand | Vct+V lime / V sand | Mass cement [g] | Mass lime [g] | Mass sand [g] | Total mass [g] |
|---|---|---|---|---|---|---|---|
| **1.3** | 0.333 | 0 | 0.333 | 4167 | 0 | 12500 | 16667 |
| **1. 4½** | 0.222 | 0 | 0.222 | 2778 | 0 | 12500 | 15278 |
| **1.6** | 0.167 | 0 | 0.167 | 2083 | 0 | 12500 | 14583 |
| **1.8** | 0.125 | 0 | 0.125 | 1563 | 0 | 12500 | 14063 |
| **1.¼.3** | 0.333 | 0.083 | 0.417 | 4167 | 413 | 12500 | 17080 |
| **1.½.4½** | 0.222 | 0.111 | 0.333 | 2778 | 551 | 12500 | 15829 |
| **1.1.6** | 0.167 | 0.167 | 0.333 | 2083 | 826 | 12500 | 15409 |
| **1.2.9** | 0.111 | 0.222 | 0.333 | 1389 | 1102 | 12500 | 14990 |

Bulk density 1450 kg/m³, using Equation 3.1a. and 3.1b.

### 3.2.2 Batching and Mixing Procedure

Comparison of the performance of mix volume proportions in bond strength requires a high degree of repeatability and control of variability during mix preparation. The standard batching and mixing procedure specified in BS 4551 was modified as follows:-

To ensure even distribution of constituent materials prior to addition of water, the dry ingredients were thoroughly mixed in a rotating drum mixer.

A proportion of the target mix water was placed into the mixing bowl, prior to adding the dry ingredients. It was found that this procedure helped to prevent a dry portion of mix ingredients, mainly finer sand particles, remaining at the base of the mixing bowl after mixing.

The time required to add the remainder of the mix water was increased to avoid water and particulate loss due to splashing.

### 3.2.3 Batching of Dry Ingredients

Batching was carried out by weight using the proportions given in Table 3.2. above. 10% additional dry ingredients were added to each batch to enable the final mass required to be completely removed from the mixer. Batching was conducted using a horizontal rotating drum mixer reserved for dry mixing. Ingredients were placed in the mixer drum in the order of sand, cement and lime (if added), and batching of each mix was repeated three times. The drum was then covered with a plastic hood to reduce dusting and loss of fine material and mixed for 10 minutes. After standing for a further 5 minutes to allow for dust to settle within the drum, mixes were weighed-out in sets of three to yield the total mass given in Table 3.2. Batches were then bagged and an airtight seal applied. The remaining 10% wastage material was brushed from the drum and stored in case further analysis was required.

### 3.2.4 Mixing

Evidence from trial mixes had previously identified that a portion of ingredients at the base of the mixing bowl was left dry after mixing. It was considered that mixing this dry portion into the mix by turning-in on the spot-board could significantly alter the required water content of this mix, since the amount of dry material could vary between mixes. Therefore 10% of the final target mixing water was placed into the bowl prior to adding dry ingredients. This water was washed around the bowl to dampen the surface so that all parts of the mix contacted the water and each mix would be consistent, regardless of whether a wet or dry mixer bowl had been used.

The dry batch was then added to the bowl and mixing was carried out for 30-seconds. It was considered that adding water over the next 30-seconds as recommended in BS 4551 did not give the mix sufficient time to absorb the mixing water and considerable amounts of splashing could occur. Therefore water was added at a steady rate over the next 60-seconds. Mixing was continued for a further 30-seconds after the addition of water to allow the initial mixing phase to be 2-minutes, consistent with BS 4551. The mix was covered with a damp cloth and allowed to stand for 10-minutes. A final mix of 60-seconds was made before the mix was turned out onto the spot-board. As a final check on the thoroughness of mixing, the mix was turned-in using a trowel on the spot-board. The mortar was then covered with a damp cloth during testing and sample preparation to reduce evaporation.

In some instances, there was still evidence of dry constituents in the base of the mixing bowl. However, with the above approach, the material at the base of the bowl was wetter than previous mixing techniques and therefore it was considered that the water content would not be significantly affected by working in this portion of the mix on the spot-board.

### 3.2.5 Assessment of Workability

Two methods were used to determine mortar consistence. These were also considered to provide a measure of mix repeatability:-

Dropping ball as outlined in BS 4551[46] section 10.

Penetrometer as outlined in DIN 4211[65].

For the purpose of this investigation the measurement of mortar flow, outlined in BS 4551 section 12, was not made due to the absence of standardised flow tables. The weight of the table top can differ from 3.2 to 6.6 kg and drop height can also vary[50]. Drysdale and Gazzola[63] determined that mortars with flows less than 110% were not sufficiently workable for good brick laying and flows of 130% did not have sufficient stiffness to support the bricks. This would indicate that flow measurement does not provide very wide margins for measurement and perhaps is not sufficiently sensitive to detect discrete changes in water content. In addition the measurement of flow takes a relatively long time and involves a considerable amount of mortar wastage. Hence readings would have been taken less frequently and at the expense of consistence and air measurements.

Trial mixes were used to determine water content for each mix designation. A target figure for consistence by dropping ball was set at 11±1 mm, which was identified during trial work as having the most suitable range of workability; this was independently confirmed by two bricklayers.

Having determined the water content required to achieve specific consistence, dropping ball and penetrometer readings were taken every five minutes corresponding to the manufacture of each sample. In addition air content by density method, outlined in BS 4551 section 13 was measured at each 5-minute time intervals, corresponding to couplets produced. Mortar from each test was returned to a separate pile on the spot-board after each measurement and was not used for couplet manufacture.

### 3.2.6 Experimental Programme

Two Fletton bricks were laid plain face to plain face to provide a uniform bed-face, without frogs or perforations. Couplets were manufactured in accordance with the procedure outlined in Section 2.2.2. 12 couplets were made at 5-minute intervals using suction adjusted units (refer Section 6.2). Mixes were repeated twice, yielding 24 samples for each mortar designation shown in Table 3.2. Corresponding consistence and air measurements were taken at each time interval. Couplets were tooled on both faces 15-minutes after manufacture and then allowed to cure in the laboratory (20°C, 40%RH) for 1-hour before being transferred to curing chamber (20°C, 80%RH).

All samples were made within 1-hour of mortar mixing, which was considered to be the upper limit for mortar spot-board life.

### 3.2.7 Materials

In order to characterise mortars in terms of bond strength it would have been beneficial to use a brick of uniform suction background. However due to the nature of variability within the material this proved impractical. Therefore it was decided to focus on producing an optimum mortar in bond strength across a range of brick suction backgrounds, similar to those encountered on site. This would allow subsequent work in Chapters 6 and 7 to identify suction parameters in relation to bond strength, based upon a representative mortar.

The bricks used throughout the entire test programme were Fletton common clay bricks with a single deep frog, produced by Hanson Brick (formerly London Brick). The bricks were smooth and supplied in kiln dried condition from The Peterborough Works in 1992. The compressive strength of the bricks was 35±2 $N/mm^2$ when tested to BS 3921[66]. The bricks had a dry density of 1745±10 $kg/m^3$, a 24-hour water absorption at 20°C of 16.4% and a suction rate range as received of 0.8 to 3.31 $kg/m^2$.min. This exceeds the maximum value of 1.5 $kg/m^2$.min given in BS5628 Part 3[67] which recommends short term immersion of such bricks on site.

As a means of limiting the range of suction, units were suction adjusted using 2-minute soak and 10-minute drain with the bed face uppermost. After suction adjustment the range was reduced to between 0.41 to 1.6 $kg/m^2$.min., for a sample of 100 units. The initial suction rates were measured in accordance with BS3921[66].

**Brick properties:**

| | |
|---|---|
| Type: | Delph autumn |
| Average compressive strength: | 35 $N/mm^2$ |
| Water absorption: | 16.4 % |
| Bulk density: | 1745 $kg/m^3$ |
| Porosity: | 29 % |

**Mortar properties:**

| | |
|---|---|
| Sand grading: | Type G |
| Sand bulk density: | 1450 $kg/m^3$ |
| Sand relative density: | 2.65 |
| OPC assumed bulk density: | 1450 $kg/m^3$ |
| Hydrated lime assumed bulk density: | 575 $kg/m^3$ |

The sand used throughout the research programme was a washed type G building sand conforming to BS 1200[49]. The sand used demonstrated high clay content, indicated by hard lumps of sand upon drying. This phenomena is described also by Harrison[60]. Cement and lime was obtained from the same source and supplier and was assumed to be of a constant nature specified by the manufacturer's quality control procedures.

### 3.2.8 Tensile Bond Testing

Direct tensile bond testing was carried out at 28 days maturity using the procedure discussed in Chapter 2, Section 2.2.5. The sample failure plane was observed and expressed as a percentage of bottom plane failures for the total number of samples.

## 3.3 Experimental Results

**Table 3.3: Average Bond Strength, Mortar Consistency and Air Content Results**

| Mortar mix | Water content [ml] | Dropping ball [mm] | Penetro-meter [mm] | Air content [%] | | Bond strength [N/mm$^2$] | Bottom plane failure [%] |
|---|---|---|---|---|---|---|---|
| | | | | | **Average** | **0.13** | |
| **1:3** | 4000 | 11.8 | 16.0 | 3.1 | S.D. | 0.023 | 100 |
| | | | | | CoV [%] | 18.0 | |
| | | | | | **Average** | **0.11** | |
| **1:4½** | 4000 | 12.6 | 27.0 | 3.0 | S.D. | 0.021 | 100 |
| | | | | | CoV [%] | 19.0 | |
| | | | | | **Average** | **0.09** | |
| **1:6** | 3880 | 11.3 | 26.0 | 3.9 | S.D. | 0.015 | 100 |
| | | | | | CoV [%] | 17.0 | |
| | | | | | **Average** | **0.07** | |
| **1:8** | 3700 | 10.3 | 19.0 | 4.4 | S.D. | 0.02 | 100 |
| | | | | | CoV [%] | 28.0 | |
| | | | | | **Average** | **0.11** | |
| **1:¼:3** | 4200 | 11.9 | 18.0 | 3.2 | S.D. | 0.027 | 100 |
| | | | | | CoV [%] | 24.0 | |
| | | | | | **Average** | **0.11** | |
| **1:½:4½** | 4000 | 11.1 | 19.0 | 3.6 | S.D. | 0.032 | 92 |
| | | | | | CoV [%] | 28.0 | |
| | | | | | **Average** | **0.13** | 100 |
| **1:1:6** | 4000 | 12.1 | 12.0 | _ | S.D. | 0.033 | |
| | | | | | CoV [%] | 25.0 | |
| | | | | | **Average** | **0.12** | |
| **1:2:9** | 4000 | 12.1 | 23.0 | 2.8 | S.D. | 0.028 | 94 |
| | | | | | CoV [%] | 24.0 | |

Couplet bond strength results show an average of two mixes for each mortar designation, producing an average bond strength for 24 couplets. In total 192 couplets were tested. Mortar consistency by dropping ball was set to 11±1 mm. Dropping ball, penetrometer and air readings are an average of first five readings.

## 3.4 Analysis of Results

Individual couplet bond strength results for all mixes in the array were analysed using a one way analysis of variance. A Tukey pairwise comparison test was performed to determine which mixes differed significantly from each other. Results are shown in Table 3.4 below.

There was a highly significant effect of mix designation on bond strength (ANOVA: $F=13.19$, $p<0.001$, d.f.=7,179).

**Table 3.4: Average Bond Strength Results Using a Constant Mass of Sand Batching Procedure**

| Volume ratio Cement:lime:sand | Average bond strength [N/mm$^2$] | | | | |
|---|---|---|---|---|---|
| 1 : 1 : 6 | 0.13 | | | | d |
| 1 : 3 | 0.13 | | | c | d |
| 1 : 2 : 9 | 0.12 | | | c | d |
| 1 : ½ : 4½ | 0.11 | | b | c | d |
| 1 : ¼ : 3 | 0.11 | | b | c | d |
| 1 : 4½ | 0.11 | | b | c | |
| 1 : 6 | 0.09 | a | b | | |
| 1 : 8 | 0.07 | a | | | |

Bond strength results joined by same letter code do not differ significantly from each other. Tukey test $p> 0.05$.

## 3.5 Interpretation and Discussion of Results

Table 3.4 shows that mortars with lime added are all significantly stronger in bond strength than cement:sand mixes, with the exception of 1:3 mortar. Bond strength increases markedly with increasing cement content in the absence of lime.

Bond strength would appear to maximise as the ratio of cementitious binder material to sand approaches 1:3, and would seem to be optimum for a 1:1:6 mortar.

Mortars with lower bond strengths have higher air content. Air content for each mix reduces during spot-board life of the mortar.

Lime mortars reflect generally a constant water content. For cement:sand mortars, water content increases with increasing cement content towards a ratio of 1:3. Water content is held relatively constant by using a constant mass of sand as shown in Figure 3.2. Any deviation in water content between mix designations, for a specific mortar consistency, is attributable to variation in amounts of cementitious binder material.

The ratio of water to binder material by volume demonstrates a linear trend when plotted against bond strength. It can be seen in Figure 3.3. that the lower the ratio of water to cementitious filler material by volume, the higher the bond strength. This trend maximises around unity, after which continued addition of cementitious filler becomes detrimental; as in the case of 1:¼:3 mortar.

The presence of a constant mass of sand in mix design means that any variation in the water content between different mix volume ratios, for a given consistency, is uniquely dependent upon the addition of cement and lime filler material. Referring to Figure 3.4, the regression lines for bond strength have been superimposed on the graph. At first examination it may be considered that the increase in lime content has only a marginal affect upon bond strength and that bond strength increases dramatically with increasing cement content. This would confirm a relationship between mortar compressive and tensile strength with bond strength dependent upon cement content. However, when considering lime as an additive, there is a marked and significant increase in bond strength between mixes where lime is added. It is arguable that this increase in bond strength could be attributed to carbonation of the lime although it is highly improbable that such a significant effect could be distinguishable by 28-days.

The hypothesis that lime additive increases bond strength is troubled by the designation (i) 1:3 cement:sand mortar achieving a high bond strength, comparable to lime mortars. However upon closer examination it can be demonstrated that all mortars which have the proportion of cementitious material to sand in the ratio of 1:3 perform well in bond strength and are significantly different to those mortars with lower mix ratios, as shown in Table 3.4.

Figure 3.4 demonstrates that towards the higher end of lime additives, there is a significant difference in bond strength between lime mixes and non-lime mixes. As the amount of lime additive is reduced to compensate for increasing cement contents, the marked differences in bond strengths reduce until the regression lines intercept. The interception for the sand used in this particular study corresponds to a ratio of cementitious material to sand of 1:3.7, suggesting that the addition of lime beyond this point has no further benefit for bond strength development.

Superposition of the regression lines for water contents necessary to produce a given consistency in Figure 3.2 follow the same trend as the regression lines for bond strength in Figure 3.4. Lime mixes hold a stable water content with a slight fluctuation for 1:¼:3. For cement sand only mixes, the water content increases with increasing cement content. The interception of the two regression lines for water occurs approximately at the ratio of 1:3.4, which could correspond to 1:3.7 found for bond strength.

This suggests that the optimum water content is achieved for a ratio of cementitious filler to sand in the region of 1:3.4. If the ratio of filler to sand exceeds 1:3.4 for the particular sand used, then the addition of water necessary to satisfy the increase in lime content becomes detrimental to bond strength, as in the case of 1:¼:3.

The above analysis is for a sand of bulk density in the region of 1450 kg/m$^3$. Sands with greater bulk densities approaching 1900 kg/m$^3$ result in lower additions of lime and cement by mass in order to achieve the equivalent volume ratios. Referring to Figure 3.2 above, it can be seen that this effect becomes more marked as the volume ratio approaches 1:3 due to the percentage of fines increasing and this in turn will cause a higher demand for water. The addition of lime required to accommodate variation in bulk density is less marked than for cement and therefore the resulting interception for water content regression lines will shift towards a ratio of 1:3.

In summary it can be argued that the addition of lime to mortar serves to compensate lowering cement contents while maintaining the volume ratio of 1:3. Water contents required to produce workability are determined by the surface area of the binder material and in order to achieve the optimum water content for workability it is necessary to maximise particle surface area and packing. The results demonstrate that the 1:3 ratio is optimum since the particle surface areas hold water levels constant.

Mixes with lower cementitious filler contents, typically the non-lime mortars, while portraying lower water contents, have increased water-cement ratios, leading to loss in strength and durability. Most importantly, mixes with low cementitious surface area do not demand high water contents and show low water retention. Subsequently less water remains adsorbed by the hydrophilic nature of the particles resulting in increased shrinkage of the mortar. Low cementitious filler mixes hold less water for a given consistence and have less retentivity to resist external suction forces.

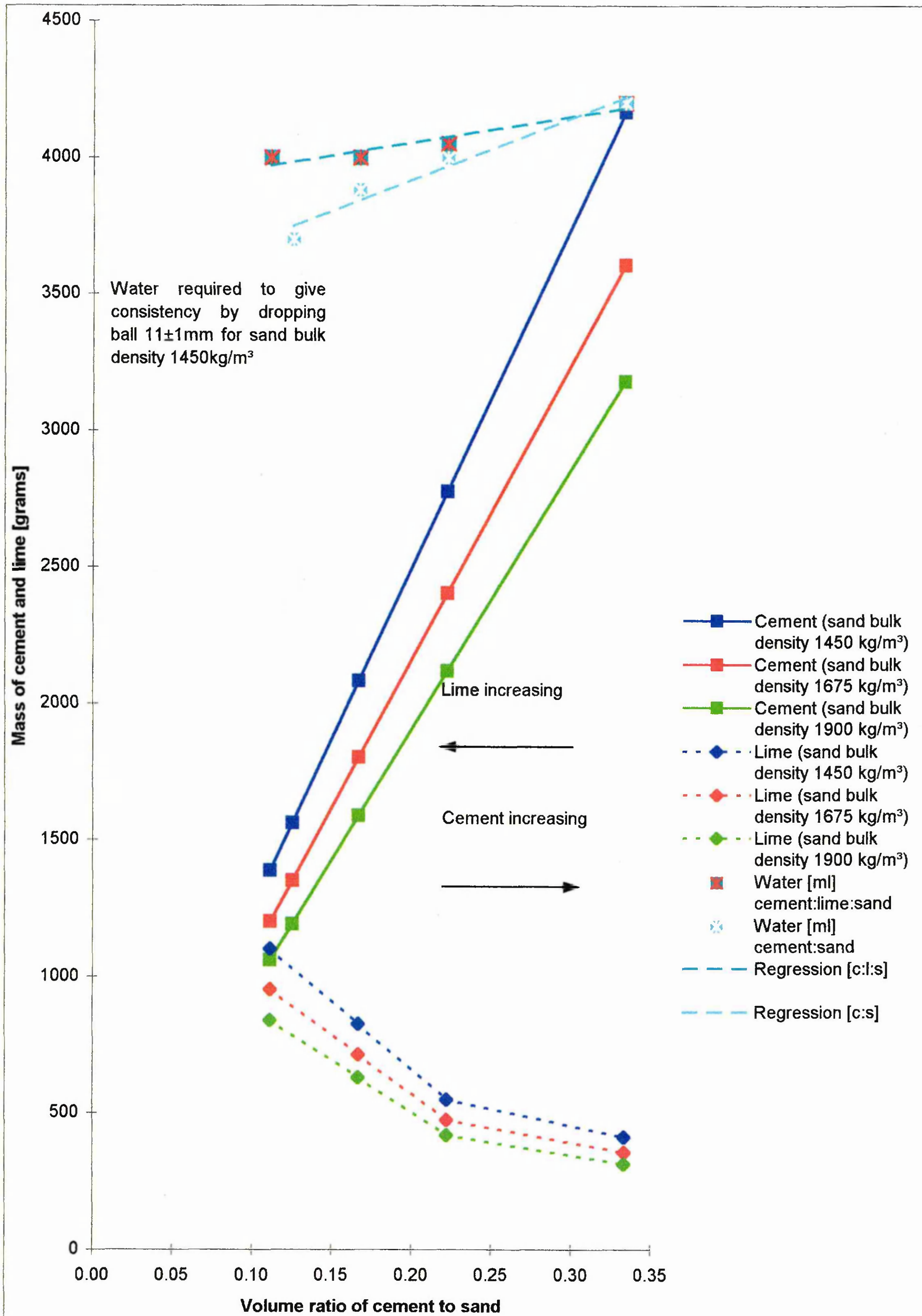



**Figure 3.2: Relationship Between Sand Bulk Density and Cementitious Material for Mix Volume Ratios Based Upon Constant Mass of Sand Mix Proportions.**

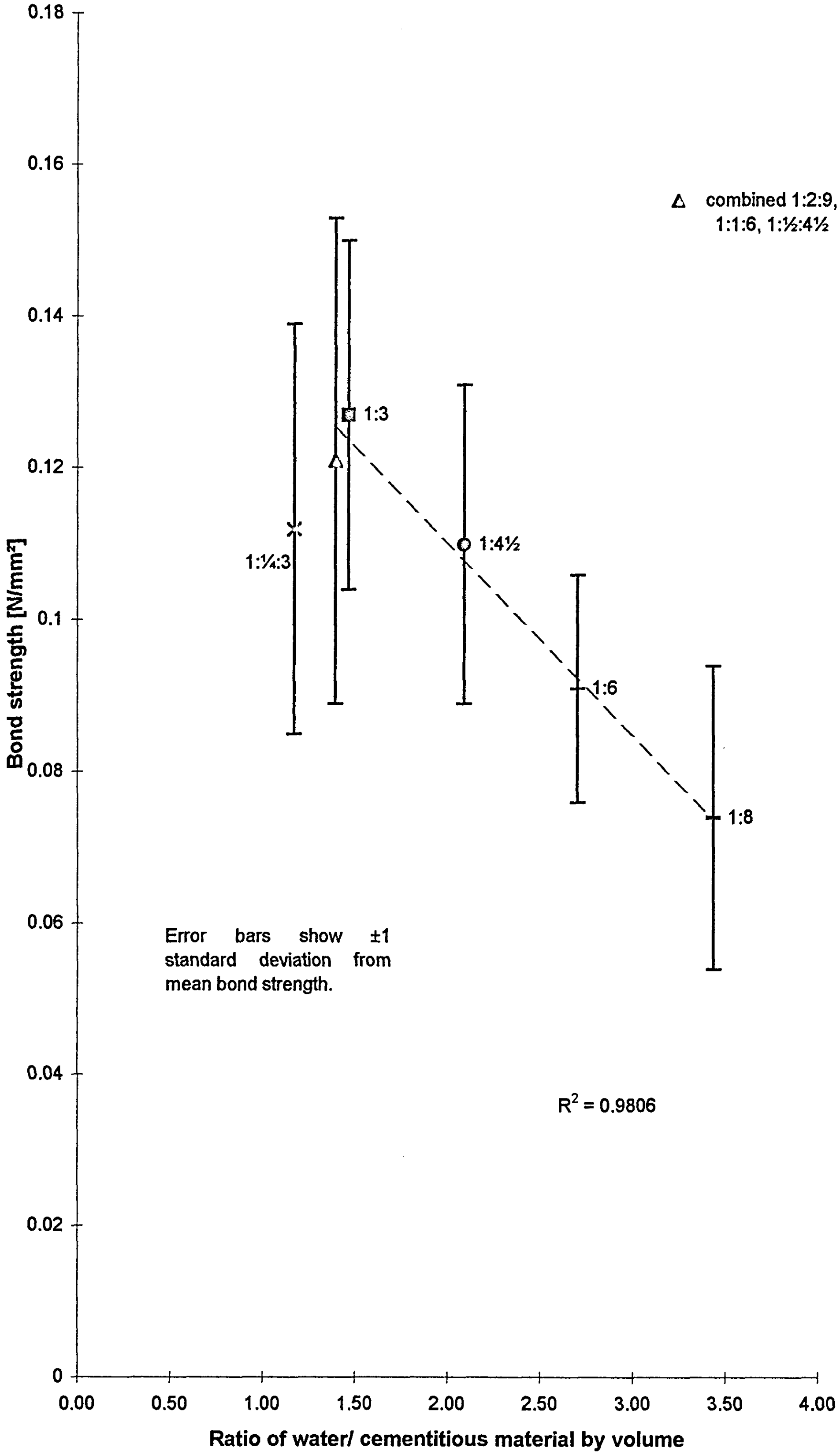



**Figure 3.3: Relationship Between Ratio of Water to Cementitious Material by Volume Against Bond Strength.**

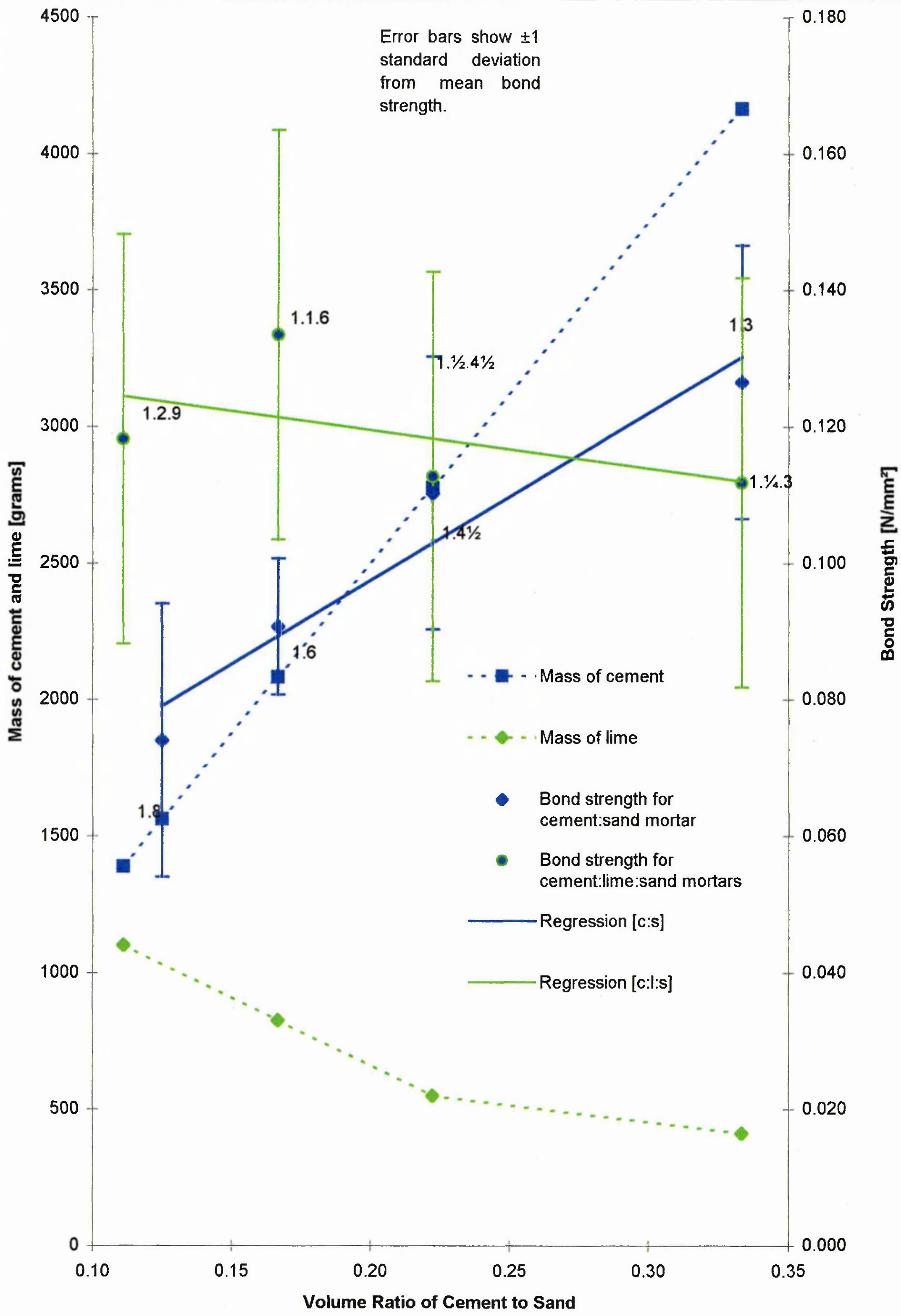



**Figure 3.4: Relationship Between Mix Volume Ratio and Tensile Bond Strength for a Constant Mass of Sand (bulk density 1450 kg/m³).**

## 3.6 Concluding Remarks

The traditional approach to mortar batching using a proportion of fine material such as cement or cement and lime, equal to about one-third the volume of a typical sand, has been reinforced by this work.

Filling the voids in the sand with cementitious material requires less water to achieve a given workability. Subsequently the water required for workability purposes is minimised and is matched to the proportion of hydrophilic material which optimises retentivity of the mix. Optimum packing results in reduced excess mix water, thereby minimising volumetric shrinkage.

The hypothesis to be developed in subsequent chapters supports the view that volumetric shrinkage is a critical controlling parameter in the development of bond strength, while having secondary implications on mortar porosity and durability.

The contribution of cement and lime does not appear to have any particular significance in relation to bond strength provided that their combined proportions maintain a 1:3 ratio with the sand. The volume of water should roughly match the volume of cementitious material, and this ratio has been shown to provide suitable workability. A beneficial factor of this finding is that the addition of target mix water could be specified for laboratory based mixes, providing batching is carried out by volume and the final quantity of cementitious material known.

The implications of these findings are that mix designations with a ratio of 1:3 cementitious binder to sand all support similar bond strength development. Mixes which fall outside the 1:3 ratio such as 1:¼:3, 1:4½, 1:6 and 1:8 may be best suited for use with masonry units of lower or higher than normal suction backgrounds.

Cement contents can be specified, dependent upon the required compressive strength and durability performance of the mortar, without adversely affecting bond strength performance, provided that lime or an inert filler is used to maintain the 1:3 ratio. In the past there has been a tendency to over compensate with cement in the belief that a high cement content made the mortar and hence the joint less permeable. Recently however the emphasis has shifted towards using weaker, less stiff mixes, particularly for re-pointing purposes.

The Building Research Establishment[68] have promoted the use of a general purpose mix, which is an air entrained 1:1:5½ cement:lime:sand mix which is similar in constitution to the 1:1:6 optimum mix found for bond strength in this investigation.

The conclusions of this work do not dismiss the influence of cement hydration upon bond strength development over time, but argue that the initial contribution to bond is dictated during the early period of joint formation and is characterised by achieving the optimum compatibility between water content of the mix for workability, water retentivity of the mortar and suction profile of the unit, in order to reduce longer term shrinkage. It is probable, as suggested by Hogberg[64], that there is an optimum mortar which can be used, irrespective of brick background suction.

Drysdale and Gazzola[63] argue that while compressive strength may be the controlling factor for some situations, it is known that the compressive strength of brick prisms is not highly sensitive to the compressive strength of the mortar. Therefore it could be argued that, in most cases, the content of cementitious material required for workability of the fresh mortar and durability of the hardened mortar will be more than adequate for compressive strength requirements.

Suggestions that bond strength performance may be related to compressive and tensile strengths of the mortar, dictated by cement content, is dismissed by this piece of work. It is concluded that bond strength is dependent not upon the properties of the mortar bed but rather upon the characteristics of a thin zone of mortar at the bonding interface. Previous methods of tensile testing which employ flexural bending in their analysis take into consideration the overall properties of the mortar bed, such as compressive strength and stiffness, tensile strength and to a lesser degree, bond strength.

The work outlined in this chapter has focused upon controlling variability in mix design and has promoted repeatability. Use of the constant mass sand philosophy is fundamental to understanding the behaviour of the wet mix and therefore the process of bond formation. The effect of increasing levels of cement or lime, or the interaction between the two, cannot be identified using the standard laboratory mix procedure outlined in BS 4551, since it is not possible to isolate individual variables.

Workers have identified sand type as a major contributory factor to bond strength performance. However the absence of a constant mass of sand in experimental mix designs makes it impossible to distinguish whether the sand properties alone influence bond strength performance or rather sand grading influences packing of mortar constituents which in turn dictates mix water content required to achieve a given mortar workability.

## References


17. Harrison, W.H. and Bowler, G.K. Aspects of Mortar Durability. British Ceram. T.& J., 89, (3), 1990.

31. De Vekey, R.C. The Influence of Mortar Type on the Flexural Strength of Masonry. Proceedings of the 7th International Brick Masonry Conference, Melbourne, Australia, Feb. 1985.

39. Palmer, L.A., Parsons, D.A. A Study of the Properties of Mortars and Bricks and Their Relation To Bond. National Bureau of Standards Journal, Rp 683. 1934.

46. British Standards Institution. Methods of Testing Mortars, Screeds and Plasters. BS4551. 1980

47. Pyle, M. Aggregates for Mortars. Geological Society Engineering Geology Special Publication No. 1,pp 145-157. 1985.

48. Bessey, G.E. Current Developments Affecting the Design and Use of Mortars for Building Purposes. Journal of Applied Chemistry, Vol 16, November 1966.

49. British Standards Institution. Building Sands From Natural Sources. BS1199 and 1200: 1976.

50. Bowler, G.K., Jackson, P.J., Monk, M.G. The Measurement of Workability. Masonry International vol. 10, No 1, 1996.

51. ReUnion Internationale Des Laboratoires D'Essais et de Recherches Sur les Materiaux et les Constructions. Testing Methods for Mortars and Renderings. RILEM Recommendations MR 1-21,1982.

52. McIntosh, J.D. Specifying the Quality of Bedding Mortars. Cement and Concrete Association.

53. Watstein, D., Seese, N.A. Properties of Masonry Mortars of Several Compositions. ASTM Bulletin 147, 77, 1947.

54. Hogberg, E. Influence of Moisture on Bond. Proceedings RILEM/ CIB Symposium, Moisture Problems in Buildings. Helsinki, 1965.

55. Currie, D., Sinha, B. Survey of Scottish Sands and Their Characteristics Which Affect Mortar Strength. Chemistry and Industry. Sep.1981.

56. Van Rensburg, J.J.J., Kloppers, E.H.L., Kruger, J.E. Factors Influencing the Bond Strength Between Calcium Silicate Bricks and Mortars. Pretoria, CSIR, Res. Rep. 340, 1978.

57. Benningfield, N. Aspects of Cement Based Mortars for Brickwork and Blockwork. Concrete, pp27-30. Jan. 1980.

58. Lee, H.N. The Technology of Mortar Design. Chem. and Ind. Pp114-117. 1975.

59. Bowler, G.K. Deterioration of Mortar By Chemical and Physical Action. Masonry International, Vol. 6, No 3. 1993.

60. Harrison, W.H. Durability Tests on Building Mortars: Effect of Sand Grading. BRE Paper1986.

61. Davison, J.I. Effect of Air Content on Durability of Cement:-Lime Mortars. Durability of Building Materials. pp23-24, 1982.

62. Held, L., Anderson, C. Effect of Fines Content of Sand on Tensile Bond. Proceedings of the 7th International Brick Masonry Conference. Melbourne, Australia, Feb. 1985.

63. Drysdale, R.G., Gazzola,E. Influence of Mortar Properties on the Tensile Bond Strength Of Brick Masonry. Proceedings of the 7th International Brick Masonry Conference. Melbourne, Australia, Feb. 1985.

64. Hogberg, E. Mortar Bond. National Swedish Inst. for Building Research 1967.

65. DIN 4211. Deutsches Institut FuR Normung. Putz and Mauerbinder DIN4211:1986.

66. British Standards Institution. Specification for Clay Bricks. BS3921. 1985

67. British Standards Institution. Code of Practice for Use of Masonry. Part 3, Materials and Components, Design and Workmanship. BS5628: Part 3: 1985

68. Building Research Establishment. Building Mortar. BRE Digest 362, 1991.

# FORMATIVE TENSILE BOND DEVELOPMENT

## Chapter Summary

*Work reported in Chapter 4 traces the initial bond strength development of brick-mortar couplets aged from 5-minutes to 24-hours post manufacture and identifies this as the critical period for primary bond formation. Observations identify a peak in bond strength at around 18-hours post manufacture, which is associated with the failure interface becoming predominately bottom plane. The work discussed in this chapter challenges much of the previous research on bond strength which adopts the 28-day benchmark for testing bond strength.*

# 4.0 Introduction

The conclusions presented in Chapter 3, challenge the consensus that the tensile bond strength development of brick-mortar interfaces are favoured by parameters uniquely related to the compressive or tensile properties of the mortar. Parameters deemed favourable to bond strength development have traditionally been associated with mortar properties, due to the observation that tensile bond strength development followed closely compressive and tensile strength gain of the mortar. The doctrine that the presence of cement in varying quantities contributes in some way to the ultimate bond strength, has been compounded by the application of flexural testing methods and has resulted in bond performance being assessed at 28-days, in line with concrete research.

While the 28-day flexural bond strength values give realistic indication of in-situ structural behaviour of the joint at that age, they do not assist in the identification of early bond formation processes or the longer-term performance of the brick-mortar bond interface.

Observations made during couplet manufacture in this study have demonstrated that bonds formed at 2-minutes of age are sufficient to support the mass of the lower brick and typical bond strength values at 5-minutes are of a magnitude of 10% of the 28-day bond strengths. Furthermore, it can be shown that if the bond is disturbed or broken in the first few seconds after initial formation, the joint development will be permanently impaired. This rapid initialisation of bond development cannot solely be explained on the basis of theories of cement hydration.

The above observations suggest that bond strength development is largely dependent upon the bond formation processes immediately upon contact between brick and the wet mortar. Subsequent strength gain may be attributable to the stiffening of the cement paste within the mortar matrix, but the primary formation of bond remains largely dependent upon the initial formation processes.

In order to identify the parameters which make this period of formation so critical to the development of the bond strength, it is necessary to investigate the bond strength development during the mortars transition from wet to hardened state.

This chapter examines the behaviour of bond strength development of brick-mortar couplets aged from 5-minutes up to 24-hours post manufacture. To measure the relatively low bond strength values experienced with the early age samples, a comparative test rig was developed, which tested couplets in direct tension, following a procedure similar to the test outlined in Chapter 2.

# 4.1 Discussion of Literature Relating to Bond Strength Development With Age

Chapter 2 discusses the literature relating to bond testing and suggests that flexural testing methods may lead to a false association between bond strength and mortar compressive and tensile strength. One consequence of this uncertainty is that research has tended to focus exclusively on the 28-day strengths, as a means of identifying parameters relating to bond strength performance.

Conclusions outlined by many workers and detailed in Chapter 4, as to the long term mechanism of bond development, fail to explain the reasons for the rapid formation of the bond within the first few minutes after contact between the brick and the mortar.

Perhaps one explanation for the absence of research investigating bond in the primary stages of formation is due to the available testing methodology. Magnitude of bond strengths experienced during the initial stages require very sensitive methods of measurement. In addition, mortar in the fresh or wet state does not develop sufficient compressive strength to provide a compression zone necessary to mobilise a tensile failure at the bond interface during flexural testing.

Baker[34] reports that there is an assumption amongst workers that the flexural bond strength of brickwork increases with time, as the hydration of the cement becomes more complete. Baker conducted bond wrench tests on single joint specimens and determined that no clear relationship existed between flexural-bond strength and age.

Baker observed that a reduction in flexural strength could sometimes occur as the age of the specimen increased. Baker's previous work with Franken's[69] identified that the greatest bond strengths could occur as early as 3 to 7 days post construction. Baker attributes this erratic behaviour to shrinkage cracking in the outer portion of the joint, reducing the section modulus. He argues that if shrinkage cracks extend 5-mm in from the surface of the joint an apparent reduction in strength would be measured. Baker confirms that flexural testing is dependent upon the strength of material in the outer portion of the joint and draws attention to the work of Isberner[35], who reported that hydration of cement will cease if the relative humidity within the mortar falls below 85%. Consequently Isberner concluded that hydration in the outer portion of the joint ceased 3-days post construction and mortar at quarter depth and in the centre of the joint stopped hydrating approximately 12 and 15 days respectively. Baker also considered that hydrated lime does not harden in the presence of water but requires carbon dioxide for carbonation to occur. He reports that specimens made with lime mortar, exhibited virtually zero bond strength when cured under water, but then gained strength at the same rate as air cured samples when cured in air. Baker concludes that shrinkage, hydration of cement and carbonation of lime all have profound influence upon the measurement and development of flexural bond strength with time. The influence of Baker's findings upon the measurement of bond strength are reported in detail in the discussion of literature relating to tensile bond testing in Chapter 2. The findings further demonstrate that the methods of testing for tensile bond strength hold important implications for the identification of bond performance parameters.

De Vitis, Page and Lawrence[70] endorse the view held by many workers, that masonry bond strength is expected to be a function of age, with initial bond being established as soon as the unit is laid and the strength then increasing as the cementitious material in the mortar hydrates.

Aside from the influence of component materials, the authors argue that changes in ambient temperature and humidity can affect the rate of hydration of cement in the mortar and at the brick-mortar interface and thus directly influence the bond strength. They note that previous research[71,72,45] in this area has produced conflicting results with some researchers observing a progressive increase in bond strength with time, while others found a peak in bond, followed by a marginal decrease.

The authors report that the age at which bond strength tests are conducted, vary from country to country. The Australian Masonry Code AS3700[20] provides for masonry to be tested in flexural bending at the age of 7-days post manufacture; the 7-day value is considered to then represent the characteristic bond strength. The authors concern is that given the preconception that bond strength increases with time and reaches a steady strength around 28-days, in line with concrete technology, design assumptions based on 7-day strength could provide an overly cautious factor of safety. The authors conducted flexural bond strength tests using the bond wrench on stack bonded specimens, built and tested in accordance with AS 3700[20]. Tests were conducted at two different stages; short-term tests were carried out on samples aged between 1-hour and 7-days post manufacture and longer-term tests on samples aged from 7-days to 6-months. Masonry piers were constructed from clay, concrete or calcium silicate bricks, laid with two types of mortar. A 1:1:6 cement:lime:sand mortar was used with clay bricks and 1:5 cement: sand with a methylcellulose water thickener (Dynex) with the concrete and calcium silicate units. The masonry was constructed as two high stack bonded piers for the short-term tests and as three high stack bonded piers for the long-term tests; each treatment being replicated 10 times.

The short-term tests had a range of variance between 9% and 33%. In all cases the mean bond strength increased with age. For the clay bricks the bond strength showed a rapid increase in strength in the first 8-hours, achieving approximately 30% of the 7-day strength at this time and then demonstrating a reasonably steady uniform strength gain up to 7-days. In contrast, the concrete and calcium silicate unit strength curves levelled-off between 1-3 days.

The long-term strength tests showed a general increase in strength up to 180-days. However, a large temporary apparent reduction in bond strength was observed at 28-days for the concrete masonry and a minor reduction in strength was seen to occur at 14-days for the extruded clay bricks and at 6-months for both the concrete and calcium silicate units. Both concrete and calcium silicate units exhibited a general decline in bond strength post 6-months. The authors attribute this temporary loss of strength of the concrete unit at 28-days to the porosity of the concrete unit, allowing large amount of water to evaporate when the units were uncovered, temporarily slowing down the hydration process. Once a new equilibrium moisture condition had been reached, this trend would be expected to cease. The authors reported coefficient of variation (CoV) values for the longer-term tests of 11% to 30%.

The authors conclude that these findings have direct implications for both the Australian Standards AS3700, which test bond strengths at 7-days and for European, United States and Canadian structural codes which test bond strength at 28-days.

Interestingly, the authors report the nature of the failure plane of the samples. They observed that for the clay bricks, the failure locations were distributed between the top and bottom interfaces. However for concrete and calcium silicate units, there was a clear trend for failure to occur at the top interface of the joint. They attribute this to the less effective hydration of the cementitious products or lack of cement paste at the top of the mortar bed. The authors do not refer to a relationship between the nature of the failure plane and the age of the sample. The review of literature contained in Chapter 5 examines in more detail the nature of the interface failure and cites references which have reported specific interface failures.

Palmer and Parsons[73] in 1934 reported that in some cases the bond strength at 3-months was considerably greater than at 1-year.

Drysdale and Gazzola[32] determined using seven brick high stack bonded prisms that there was a significant difference in bond strength between samples tested at 365-days compared to those tested at 28-days. However, they also determined that there was no significant difference between 2-day and 28-day strengths. This demonstrates that the strength increase in bond is not significant from 2-days to 28-days, suggesting that bond formation before 2-days is critical. Drysdale and Gazzola argue that testing at 2-days age can provide a good indication of the strength at later ages and that field-testing could be conducted in time to permit faulty construction to be replaced with minimum disruption. Furthermore, the significant increase in flexural strength with time, reported by Drysdale and Gazzola, from 28 to 365 days, questions the wisdom of using 28-day strengths as a performance predictor for masonry bond strength.

Sise, Shrive and Jessop[45] also conducted bond wrench tests on five unit high prisms constructed from a range of mortars and a selection of lightweight and normal density concrete blocks and a pressed and extruded clay brick. The tests were undertaken to examine the influence of a number of parameters and were conducted at various ages between 7-days and 1-year. The results showed that for all types of units laid with 1:1:6 cement:lime:sand mortar, there was a rapid rise in bond strength over the first 7-days. Most samples reached established bond strengths, with the exception of the extruded clay unit, which depicted continuing strength gain with time. Units laid with ½:1:4½ OPC: Masonry cement: sand mortar showed a dramatic and steady decline in bond strength over the same duration, with 1-year old samples returning approximately between 10 and 20% of the 7-day optimum strength. The authors reported variation of results between 4.0% and 25.4% and remarked that the higher CoV's occurred for tests at older ages.

Arora and Hodgkinson[74] observed an apparent increase of 80% in the flexural strength for walls constructed from a 1:1:6 mortar, subject to mild exposure over a period of 10-years. For walls of 1: ¼:3 mortar, they observed no such marked increase in flexural strength. The authors suggest that one explanation for the apparent difference in behaviour is the amount of shrinkage cracking at the brick-mortar interface induced by shrinkage in the stronger mortar. Interestingly, the authors report that some wallette samples cut from existing buildings, failed in the joint parallel to the bed joint during transit. These apparent joint failures are attributed to accidental damage and are not recorded as zero bond strength. Work in the following chapter, suggests that continued drying shrinkage, over time, may actually rupture the bond and that values of zero bond are equally as valid and should not be omitted from results unless it can be clearly established that the bond has been physically damaged by factors external to the bond formation process.

Literature available on the bond strength formation with time is limited and there is still disparity between findings. While some workers report a continued increase in strength with time, others have witnessed a peak in bond strength before 28-days. It is apparent that the particular testing methods adopted will yield different results. Flexural testing, which relies heavily on the formation of mortar's compressive strength to form a "hinge", will show a continuation in flexural strength, associated with cement hydration processes. In addition, the geometry of flexural test samples, in particular wallettes, may mask any bonds that have failed prematurely, since there are usually more than a single brick bed face across any particular bed-joint. Furthermore, wallette or stack bonded pier joints, which may have failed naturally during curing, are more likely to be attributed to accidental damage, due to their vulnerability to the testing procedure. Consequently, joints, which may have failed as a direct consequence of the bonding process, could be excluded from the results. On the other hand, direct tested samples, which are more readily handled, may not demonstrate failure until they are physically loaded. Consequently, zero or low values of bond strength are more likely to be recorded. Comparison of results between samples tested by the direct and the flexural testing approach, may show bias. Flexural testing methods may lead to the impression of continued strength gain with time and may conceal the disruption in bond caused by long-term drying shrinkage.

## 4.2 Methodology of Initial Tensile Bond Testing

### 4.2.1 Development of Early Tensile Test Rig

Due to the sensitive nature of bond strengths in the early stages of formation, it was necessary to develop apparatus which could support the couplet during testing and apply a range of loads.

Figure 4.1 shows the apparatus used for testing the early bond strength of brick-mortar interfaces in direct tension.

In anticipation of the low magnitude of bond strength values in the early stages of testing, it was decided to apply a physical load using a mass of sand rather than through mechanical or electronic means; in this way, the uncertainty of measurement of such a small value was controlled. The load was applied using a lever arm in the ratio of 1:3, to amplify the quantity of sand required to induce joint failure.

The test apparatus consisted of a steel HI PLAN frame, which supported and clamped the lower unit of the couplet firmly. The loading cradle for the upper unit was adopted from the Sheffield Hallam University test rig, described in Chapter 2 and shown in Plate 4.1. The loading cradle was suspended from a wire cable, which was attached to the loading beam as shown in Plate 4.2. Attached to the other end of the loading beam was a large container suspended by a wire cable. The loading beam was pivoted using a knife edge fulcrum at one-quarter length in order to provide the 1:3 lever arm. The loading beam was equipped with a moving counter weight to ensure that the test system was in equilibrium between the loading cradle and the empty container, prior to loading.

There was concern that the load would not be applied truly vertically, due to the pivoting nature of the lever arm. To compensate for this, the wire supporting the loading cradle was kept vertical by the use of a small pulley shown in Plate 4.2. Since the wire only contacted the pulley once the loading arm began to rotate, it was considered that effects of friction could be ignored as the brittle nature of joint failure meant that the bond failed almost immediately deflection of the system occurred.

For comparative purposes, care was taken to ensure that the loading arrangement was compatible with the main direct tensile test detailed in Chapter 2. Nevertheless, it was recognised that the Sheffield Hallam test uses couplets which have established bond strength, and therefore the lower brick can be suspended from the joint. For early bond strength tests, this was not possible since the bond strength is not sufficiently developed to support the weight of the lower unit. Consequently, a different loading cradle was used to clamp the lower unit to the support frame as shown in Plate 4.1. The difference in the loading arrangements between the two tests was not considered to be influential, since it was determined in Section 2.6.5 that the loading cradle does not have significant influence upon the magnitude of the recorded bond strength or failure plane.

### 4.2.2 Couplet Manufacture

Test couplets where manufactured following the procedure outlined in Section 2.2.2. using Fletton bricks described in Section 3.2.7 and a 1:1:5½ cement:lime:sand mix with 3720ml water content and 4ml of air entrainer, following the batching and mixing procedure in Section 3.2. Consistence by dropping ball was maintained at 11±1mm and the average air content of the mortar was 13%, taken by the density method.

It was decided to adopt a 1:1:5½ air entrained mix following the BRE publication in 1991[68], recommending a single general purpose mix for both interior and exterior use. Trial mixes tested at 28-days, determined that the universal mix did not differ significantly from the optimum 1:1:6 mix reported in the constant mass sand experimentation in Chapter 3.

Due to time constraints for testing samples from 5-minutes of age, it was not practical to cure the samples in the curing chamber. In order to provide continuity with supporting work, samples were manufactured and cured in a controlled environment under ambient laboratory conditions (22°C, 40% RH). Couplets were not pointed using bucket handle pointing as for other samples, but were struck flush on both faces to avoid disturbing the bond formation process at the early stage.

### 4.2.3 Test Procedure

The test couplet was clamped in position by the lower unit approximately 1-minute prior to the specified test time. The lever arm was then slightly over balanced by putting a small quantity of sand in the loading container; this was done to allow the loading cradle to be positioned correctly and ensure that there would be no sudden impact upon commencement of loading. Dry, sieved sand was then poured into a funnel equipped with a flexible nozzle and in order to provide a consistent loading rate the funnel was kept full of sand throughout the test. Upon joint failure, the sand flow was stopped immediately and the container and sand removed. The mass of sand required to induce joint failure was then weighed to an accuracy of ± 1-gram. The mass of the container was subtracted from the sand mass and an ultimate failure load was obtained by multiplying by a factor of 3 for the lever arm; the load was then converted to force in Newton's. The failure plane of each sample was also recorded and photographed.

### 4.2.4 Experimental Programme

Samples were tested in direct tension at 5, 15, 30 minutes and 1, 2, 3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22 and 24 hours after manufacture. Couplets were manufactured every two minutes. In order to carry out testing, which could take up to five minutes, the samples had to be made in rotation. Generally the first set of mixes contained samples to be tested within the first hour after manufacture; the second mix contained samples to be tested between 1 and 10 hours and the third set of mixes contained samples to be tested between 10 and 24 hours. Care was taken to ensure that mixes did not differ in consistency, by following the procedure outlined in Section 3.2. In general, each sample age was represented by three replicates per mix and mixes were repeated three times. Therefore the minimum representation for samples tested at any particular age was nine, with some samples having larger representation.

The loading rate was measured by recording the mass of sand flowing in 60-seconds and a conversion made for the lever arm. The calculated loading rate was 154.2 N/s ±1.34.

The bond strength capacity of the early tensile rig was limited for older samples, which demonstrated higher bond strengths, due to deflection encountered in the lever arm and the size of the sand container. Therefore, the adapted rig was correlated with the Sheffield Hallam rig, to facilitate comparison between early bond strength values and mature bond strength values, 24-hour post manufacture. A comparison between the early tensile bond test and the Sheffield Hallam test was conducted using couplets aged 24-hours, which reflected bond strength near the upper limits of capacity of the early test rig.

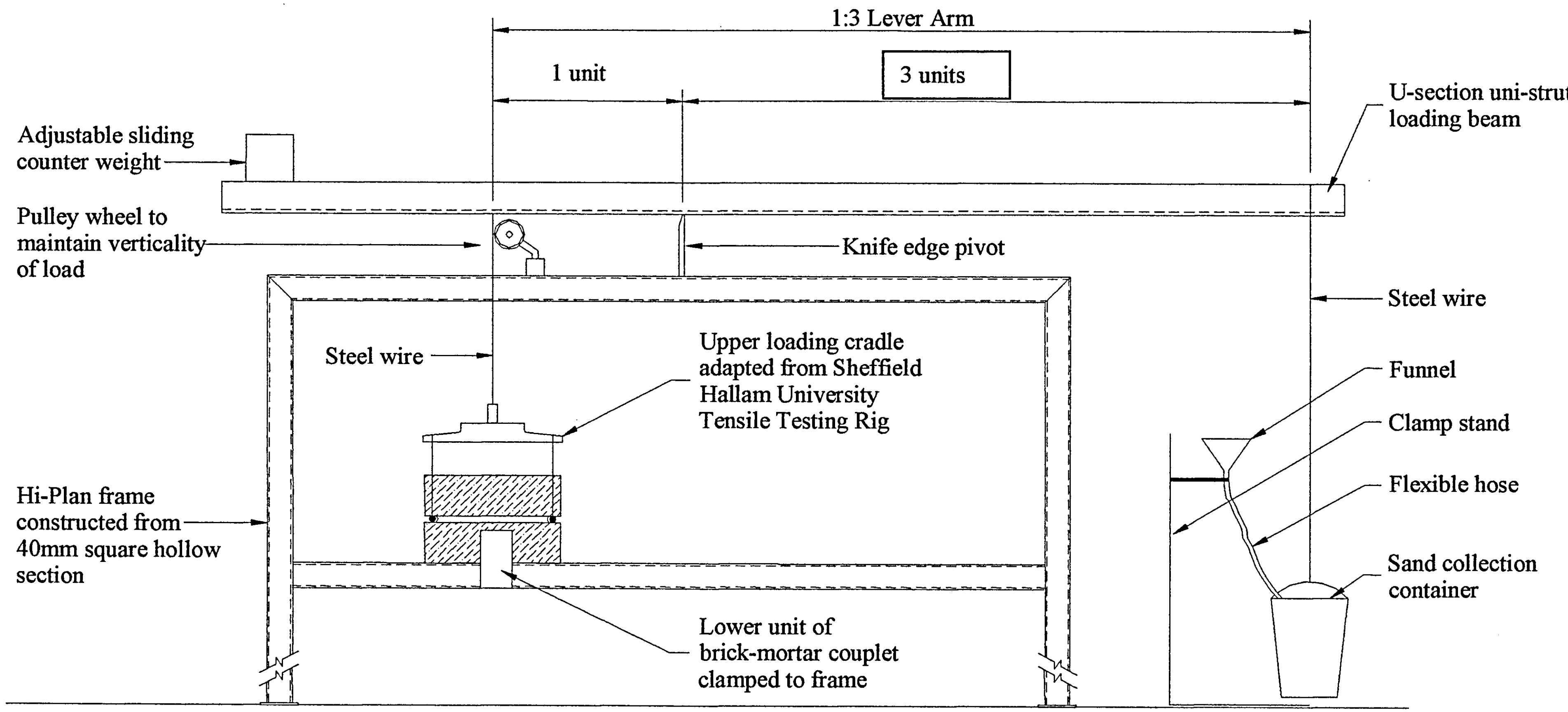



**Figure 4.1 : Showing Early Bond Strength Tensile Testing Apparatus** (not to scale)

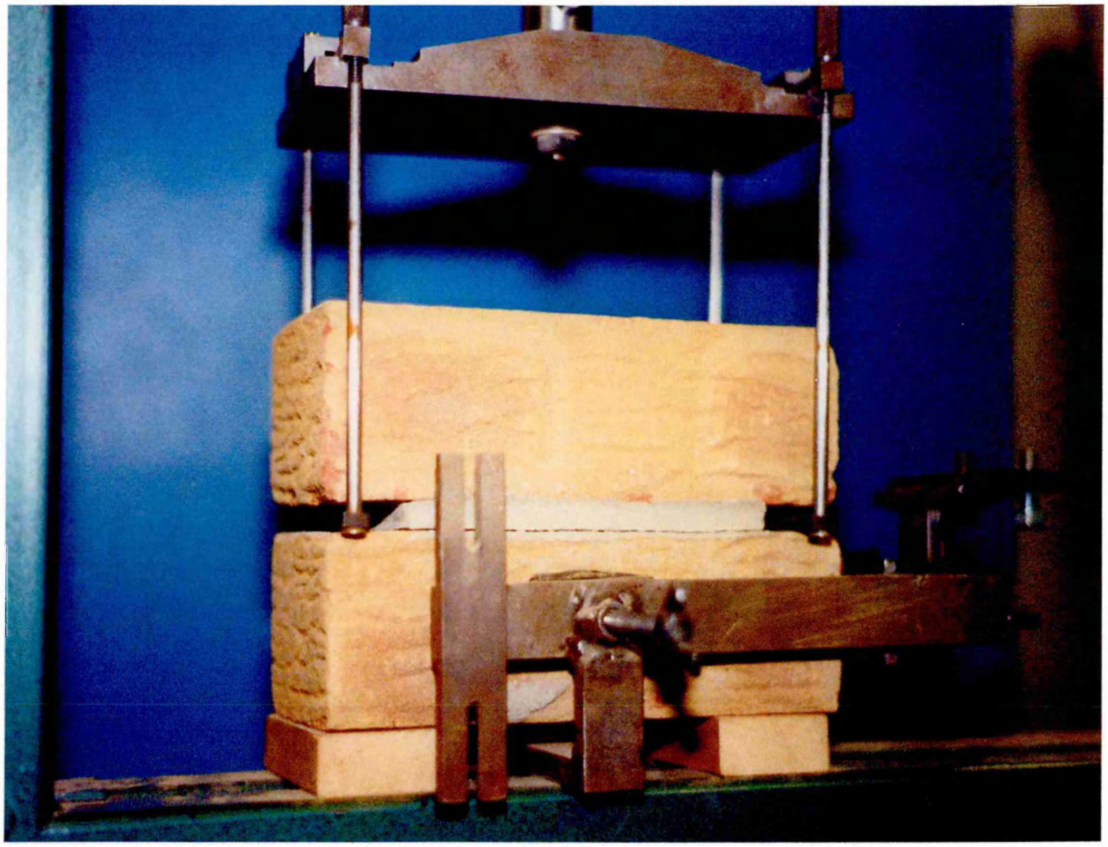


**Plate 4.1: Showing loading cradle for Early Tensile Test Rig**

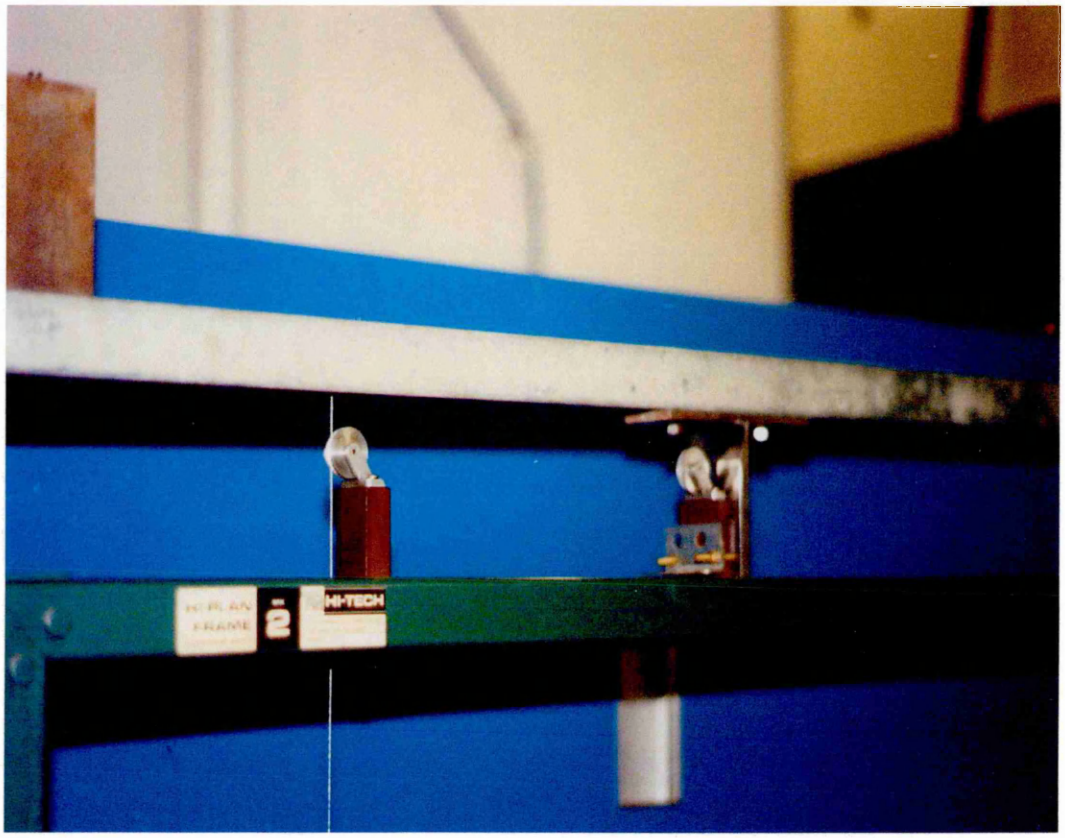



**Plate 4.2: Showing lever arm arrangement for Early Tensile Test Rig**

## 4.3 Experimental Results

Table 4.1 shows in summary, the mean bond strength values for a minimum of 9 replicates measured between 5-minutes and 24-hours after couplet manufacture. This information is also shown graphically in Figure 4.2.

**Table 4.1: Early Direct Tensile Bond Strength Results**

| Test age | Average bond strength [N/mm²] | Standard deviation | Coefficient of variation [%] |
|---|---|---|---|
| 5 min | 0.01 | 0.0026 | 30.0 |
| 15 min | 0.01 | 0.003 | 33.0 |
| 30 min | 0.01 | 0.0033 | 31.0 |
| 1 hour | 0.01 | 0.0039 | 35.0 |
| 2 hours | 0.01 | 0.0040 | 30.0 |
| 3 hours | 0.02 | 0.0042 | 26.0 |
| 4 hours | 0.02 | 0.0071 | 33.0 |
| 6 hours | 0.03 | 0.0060 | 23.0 |
| 8 hours | 0.04 | 0.0077 | 27.0 |
| 10 hours | 0.03 | 0.0099 | 29.0 |
| 12 hours | 0.04 | 0.0057 | 14.0 |
| 14 hours | 0.04 | 0.0062 | 16.0 |
| 16 hours | 0.05 | 0.0054 | 11.0 |
| 18 hours | 0.04 | 0.0127 | 29.0 |
| 20 hours | 0.05 | 0.0078 | 17.0 |
| 22 hours | 0.04 | 0.0109 | 27.0 |
| 24 hours | 0.03 | 0.0105 | 32.0 |

Bond strength values are for an average of 9 replicates

Table 4.2 below, compares the 24-hour bond strength values measured using the early tensile test rig and the Sheffield Hallam direct tensile test rig. The results were analysed using a two-sample t test.

<u>There was no significant difference between the means of the 24-hour bond strength values measured by the two different test procedures, ($t=-0.28$, $p>0.1$, d.f.=21).</u>

The results from the early bond strength tests can therefore be linked to bond strength values obtained at 24-hours, providing a bond strength maturity curve from manufacture up to 2-years of age. Such a curve is shown in Figure 5.4 in Chapter 5.

**Table 4.2: Comparison of 24-Hour Bond Strength Values Measured by Different Tensile Test Apparatus**

| **Sheffield Hallam direct tensile test** | | **Early tensile test** | |
|---|---|---|---|
| **Bond strength [N/mm²]** | **Failure plane** | **Bond strength [N/mm²]** | **Failure plane** |
| 0.05 | TF | 0.04 | BF |
| 0.04 | BF | 0.05 | BF |
| 0.05 | BF | 0.04 | BF |
| 0.04 | BF | 0.04 | BF |
| 0.04 | BF | 0.05 | BF |
| 0.05 | BF | 0.05 | BF |
| 0.02 | BF | 0.04 | BF |
| 0.04 | BF | 0.04 | TF |
| 0.05 | BF | 0.05 | BF |
| 0.05 | BF | 0.02 | BF |
| 0.03 | BF | 0.04 | BF |
| 0.03 | BF | 0.04 | BF |
| **0.04** | Average | **0.04** | Average |
| **0.009** | St. Dev. | **0.008** | St. Dev. |
| **23.0** | CoV [%] | **20.0** | CoV [%] |

BF = bottom plane failure, TF = top plane failure

### 4.3.1 Observations

During testing, it was observed that the failure plane formed either at the bottom interface, the top interface or a combination of the two. No failure plane occurred horizontally through the mortar. For the early bond tests, typically those tested less than 3-hours after manufacture, it was noted that the mortar bed broke into several pieces. After this time the mortar bed failed as a complete slab, breaking away from one specific interface. This phenomena is depicted by the photographs in Plates 4.3 [a-c].

The overlay in Figure 4.2 shows the percentage of lower interface failures along the Y-axis to the right.

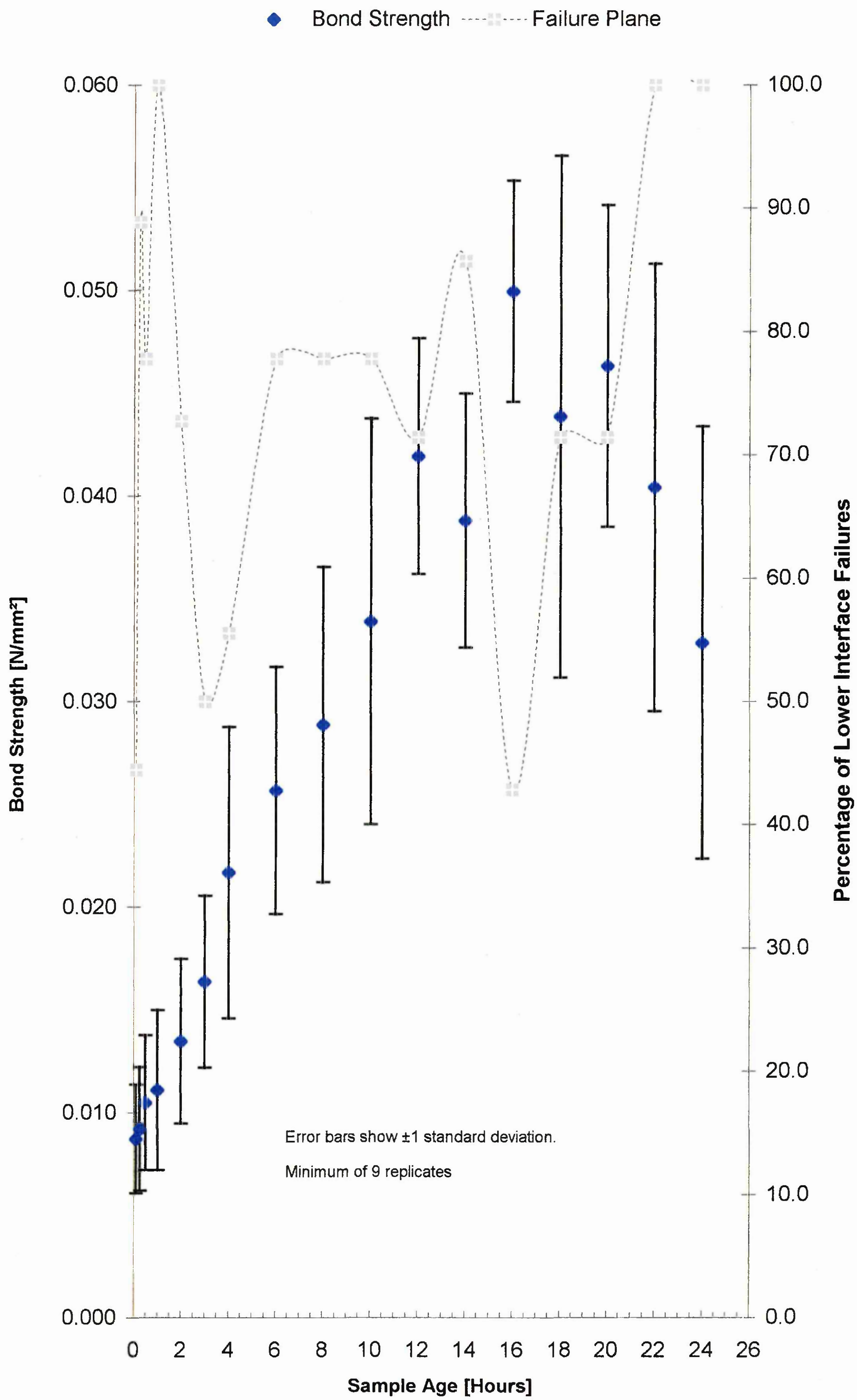



**Fig 4.2: Average Tensile Bond Strength for Couplets Aged Between 5 minutes and 24 hours.**

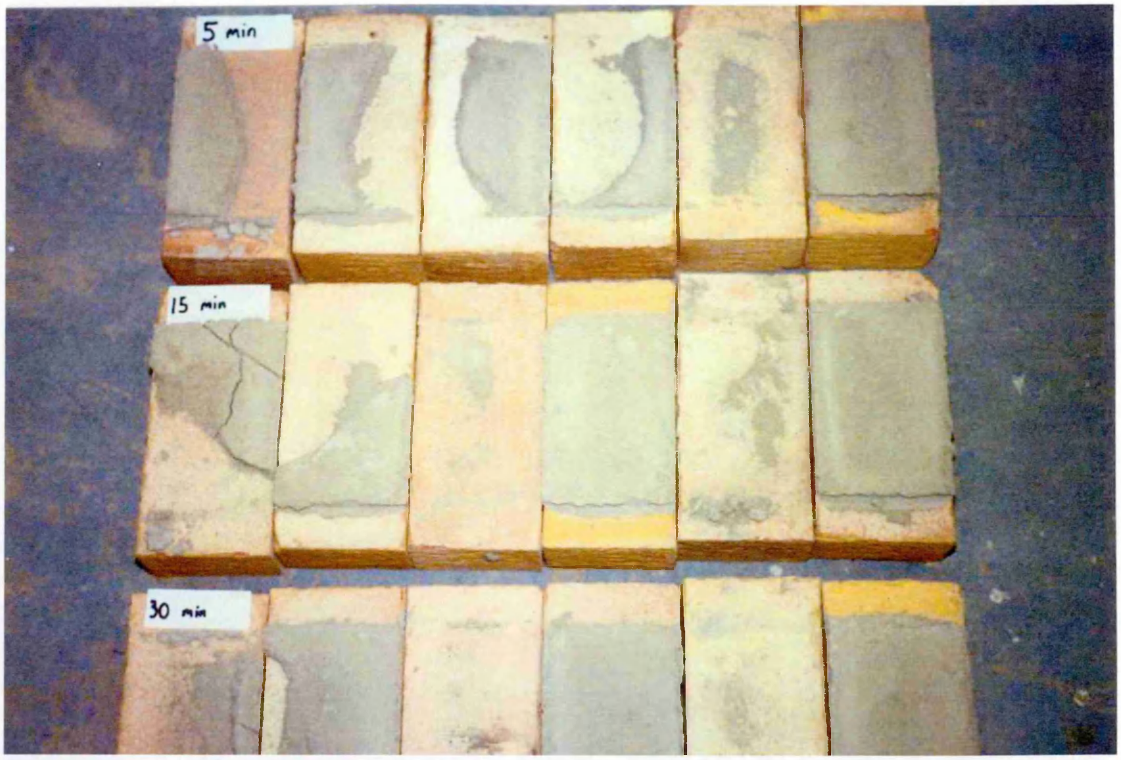



**Plate 4.3a Showing mode of bond failure at 5, 15 and 30 minutes after couplet manufacture.** (Top brick of couplet on left of photograph)

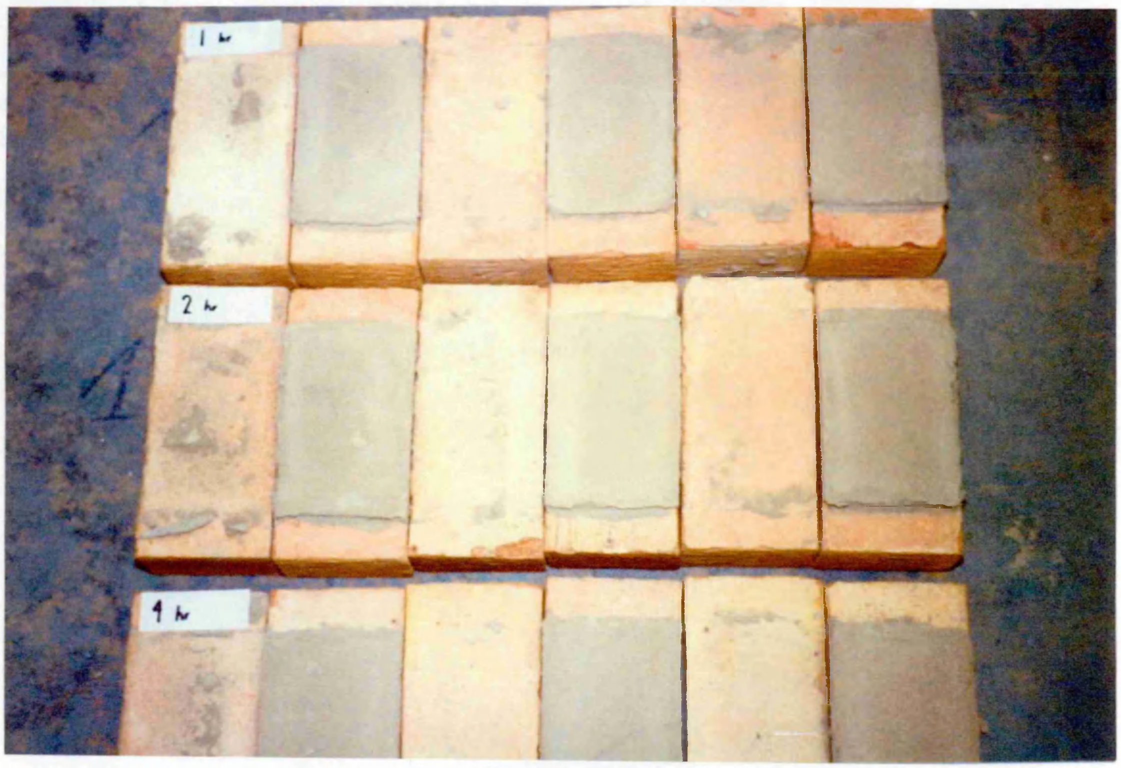


**Plate 4.3b Showing mode of bond failure at 1, 2 and 6 hours after couplet manufacture.** (Top brick of couplet shown on left of photograph)

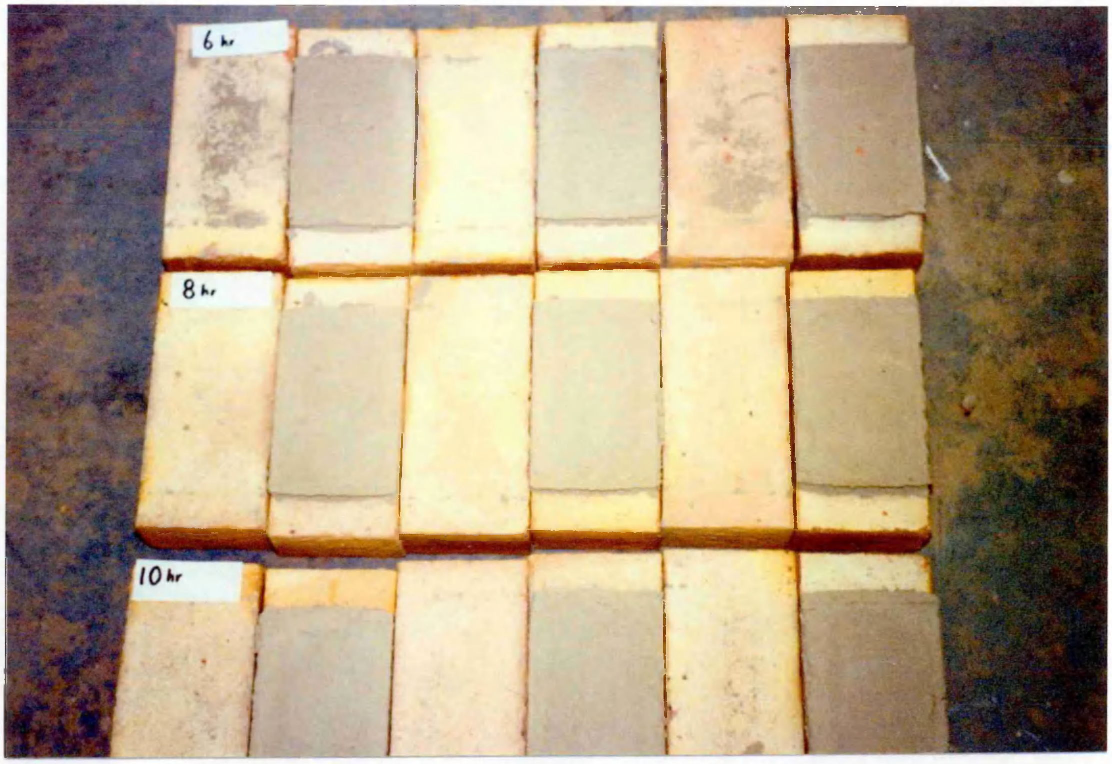



**Plate 4.3c** **Showing mode of bond failure at 6, 8 and 10 hours after couplet manufacture.** (Top brick of couplet on left of photograph)

**Plate 4.4** **Showing bonding of brick fragment to mortar bed**

## 4.4 Discussion of Results

Figure 4.1 shows a gradual increase in bond strength, reaching an apparent peak in bond strength around 18-hours after joint formation. Interestingly this peak is associated with a marked reduction in the coefficients of variation at this time. The curve then demonstrates a reduction in bond strength values between 20 and 24 hours post manufacture. Further testing described in Chapter 5 highlights that this decline does not continue and that bond strength values stabilise 24-hours after manufacture.

An interesting phenomenon to note is that the peak in the value of bond strength is associated with the lower interface becoming the predominant failure plane. This pattern of bottom plane failure continues and forms the majority of interface failures observed post 24-hours.

The apparent peaks in bond strength, associated with a reduction in the coefficients of variation, are accompanied by a change in interface failure plane. These phenomena appear to indicate that a predominant mechanism occurs around 18-hours which either gives preference to the bond formation of the upper joint, or which is detrimental to the lower interface bond.

The factors, which differentiate the bond development between the upper and lower joint, are discussed below:-

i) the placing of the mortar on the lower unit before the upper unit, giving rise to reduced plasticity of the mortar bed before the upper unit is laid.

ii) the difference between throwing the mortar onto the lower unit and the shearing or tapping action of placing the upper unit.

The method of laying of units and the associated reduction in plasticity of the mortar bed brought about by removal of the mix water by the lower brick, prior to placing the top brick, does not account for the observation that the failure plane fluctuates between the top and bottom interface over the initial bond formation period. In addition, the literature which is discussed in Section 5.2, reports primarily top plane failures, whereas this investigation showed predominantly lower plane failures post 24-hours.

The photographic evidence in Plates 4.3 a-c show that up to 3-hours in age the mortar slab demonstrates limited residual strength and undergoes significant lateral plastic shrinkage. Bond failure is brought about by the mortar slab breaking into pieces and failing simultaneously at the upper and lower interface. After 3-hours, the mortar begins to form a rigid slab and although the failure plane fluctuates between top and bottom interfaces, combined failures are no longer observed.

It could be considered that initially, mortar plastic shrinkage stresses act in unison with brick moisture expansion, as the brick absorbs free water from the mortar bed. These opposing shear forces induce a lateral "gripping" action as the mortar matrix is restrained by microscopic undulations and pores on the bedface. This mechanism is then further strengthened as the mortar bed increases in stiffness, as a consequence of cement hydration.

The process of these lateral stresses being induced at the bonding interface, coupled with intimate contact between the mortar and the bedface, could explain why the initial bonding process develops so rapidly and achieves comparatively high early strengths.

Further evidence to this proposed contribution to bond is demonstrated by small pieces of brick embedded in the mortar, upon failure, as shown in Plate 4.4. Closer examination reveals that these brick pieces are not adhered to the mortar, but rather, are retained in the mortar matrix by the lateral shrinkage of the mortar bed. This is shown in Plate 4.5. Plate 4.6, shows a similar view of the brick-mortar interface in elevation, and gives an indication of the intimacy of the bond contact.

**Plate 4.5 Showing view of embedded brick fragment in mortar matrix using an electron microscope**

**Plate 4.6 Showing elevation on brick-mortar interface using an electron microscope**

## 4.5 Conclusion

The hypothesis is presented that volumetric plastic shrinkage of the mortar bed, induced by rapid removal of the excess mix water by brick background suction, provides a mechanical lateral gripping action to the brick as the bond develops. It is considered that the immediate bonding force is generated by surface tension between the wet mortar and the brick bedface. For units with high initial rates of absorption, this surface tension bond is replaced relatively quickly by the lateral shear forces associated with mortar plastic shrinkage and to a lesser degree, by brick moisture expansion. For units with low initial rates of absorption, such as engineering bricks, this surface tension effect is sustained over a longer period. An analogy can be drawn between two glass plates which are held together by a fine film of water. A considerable direct tensile force would need to be applied in order to part these plates.

The experiment demonstrates that bond strength begins to form immediately and reaches considerable strengths, well before any form of chemical adhesion from cement bonds could be generated. Furthermore, suggestion of adhesion does not explain why the failure plane for established joints, predominantly occurs at the lower interface.

The distinct peak in bond strength values at around 18-hours after joint formation is associated with the formation of a definitive preferential failure plane. This suggests that whatever the process which benefits the bond strength development of a particular failure plane, the mechanism is progressive and forms over a considerable period of time. Chapter 5 investigates the behaviour of wet mortar-brick interfaces and examines in more detail the nature of preferential interface failures.

The findings of this investigation show that the initial 24-hours of bond development is critical. Parameters such as the use of mortar retarders or early frost exposure, could hold significant implications for the future development and performance of the interface bond. The 28-day benchmark, for testing bond strength performance, does not necessarily provide an accurate indication of the initial compatibility between materials. The work reported in Chapter 5 follows on from early tensile bond strength development and investigates the bond formation post 28-days.

## References


20. SAA Masonry Code AS3700-1988. Masonry in Buildings. Standards Association of Australia, Sydney N.S.W., 1988.

32. Drysdale, R.G., Gazzola, E. Influence of Mortar Properties on the Tensile Bond Strength of Brick Masonry. Proceedings of the 7th International Brick Masonry Conference. Melbourne, Australia, Feb. 1985.

34. Baker, L.R. Some Factors Affecting the Bond Strength of Brickwork. 5th International Brick Masonry Conference, Session 2, paper 9,1979.

35. Isberner, A.W. Properties of Masonry Cement Mortars. Designing Engineering and Constructing with Masonry Products. Gulf Publishing Company, Houston, Texas 1969.

45. Sise, A., Shrive N.G., Jessop E.L. Flexural Bond Strength of Masonry Stack Prisms. Proceedings of the British Masonry Society April 1988 No 2 pp 103-107.

68. Building Research Establishment. Building Mortar. BRE Digest 362, 1991.

69. Baker, L.R., Franken, G.L. Variability Aspects of the Flexural Strength of Brickwork. Proceedings 4th International Brick Masonry Conference, Brugges, April 1976.

70. De Vitis N., Page A.W., Lawrence S.J. Influence of Age on Masonry Bond Strength: A Preliminary Study. Masonry International, Vol. 12, No.2, 1998.

71. Kuenning, W.H. Improved Method of Testing Tensile Bond Strength of Masonry Mortars. Journal of Materials, 1, (1), 180, 1996.

72. Matthys, J.H., Grimm, C.T. Flexural Strength of Non-reinforced Brick Masonry With Age. Proceedings 5th IBMAC, Eds. J.A. Wintz and A.H. Yorkdale. Washington DC, BIA, 114, 1982.

73. Palmer, L.A., Parsons, D.A. A Study of the Properties of Mortars and Bricks and Their Relation to Bond. National Bureau of Standards Journal, Rp 6831934.

74. Arora, S.K., Hodkinson, H.R. Durability of Brickwork: Change in Flexural Strength With Time. British Ceramic Research Association.

# LONG TERM TENSILE BOND PERFORMANCE

## Chapter Summary

*Chapter 5 leads on from the early tensile bond strength work reported in Chapter 4 and considers the secondary bonding phase of mortar in the hardened state. A bond strength development curve is traced from 2-days to 98-days (14-weeks) and establishes that bond strength values plateau around 60-days. Bond strengths observed at 2-years of age show that 60% of samples reach a maximum level of bond, while the remainder fail during curing, returning zero bond strength values.*

*The susceptibility of the bond development to the curing environment is examined by exposing samples to different regimes of climatic simulation at various points along the bond formation curve.*

*The significance of assessing characteristic flexural strength, using a benchmark of 28-days, is examined.*

# 5.0 Introduction

Chapter 4 has identified that a formative bonding process occurs during the mortars transition from the wet to dry state, considered to take place within the initial 24-hours post manufacture of the joint. It is presented that this initial formation period is pivotal to the long-term performance of the interface bond. However, examining bond strength development in the early stage does not characterise the subsequent and sustained strength development of the interface bond, once the mortar has achieved its hardened form.

Long-term strength properties of mortar have traditionally been predicted by the use of 28-day test values. It has generally been postulated that bond strength continues to develop post 28-days, at a much reduced rate, in a manner similar to compressive and tensile strength development of the mortar.

Observations made during this programme of research have shown that samples tested at 2-years of age demonstrate an increasingly high proportion of interface bonds which have failed during curing, returning zero values of bond strength. The phenomena that the bond strength mechanism may be disrupted over time is highly significant when considering the future strength and durability of the finished brickwork. Such failures may be masked in the finished brickwork but may give rise to planes of weakness in flexural strength or permit isolated water penetration between the brick-mortar interface.

Unfortunately, 28-day characteristic flexural strengths, as specified in BS5628: Part 1[16], or alternative forms of bond testing carried out at this age, may not readily identify such potential disruption of the bond at later stages. It is postulated that bond strength performance criteria cannot rely upon characteristic 28-day values for future projection, since the direction of strength development remains conjectural.

The assumption that bond strength develops in a similar manner to that of the compressive strength of the mortar has been challenged in the preceding chapters.

Chapter 2 demonstrated that the nature of flexural strength testing may lead to the mistaken perception that tensile bond development is synchronous with mortar compressive and tensile strengths. Chapter 3 refutes this concept and demonstrates that cement content alone, is not necessarily a significant factor when considering brick-mortar bond strength performance. The rapid formation of the initial bond and the failure plane mechanism which have been observed previously, in Chapter 4, have demonstrated that there may be additional parameters at work which contribute to the bond development.

In order to examine more closely the formation of bond with time, a bond strength development curve was plotted for samples aged between 2-days and 98-days (14-weeks). In order to address the significance of using 28-day characteristic strength as a performance indicator, the susceptibility of the bond formation mechanism to disruption over this period was investigated by subjecting samples of various pre-cure ages to freezing. Samples were then tested for bond strength at various post-cure ages.

It was considered that by freezing the samples, the relative humidity within the mortar joint and surrounding area would be reduced below the point where cement hydration would cease; identified by Isberner[35] to be below 85% relative humidity. It was considered that inhibition of the cement hydration process at various phases would have the effect of either suspending or retarding the contribution of the cement in the bond formation process. Comparison of the treatment sample mean bond strength with those of normally cured control samples would therefore indicate whether there was an impairment of bond development or a suspension in strength gain. Furthermore, by subjecting early age samples to suspension of the bonding process, by freezing, provides an indication of the vulnerability of the bond strength of newly laid masonry to the effects of early frost action.

# 5.1 Discussion of Literature Relating to the Mechanism of Bond Strength Development

Despite the general interest in the tensile bond strength of brick-mortar interfaces, few workers have attempted to explain the mechanism of its formation. Workers writing in The 1950's have tendered their hypotheses to explain the nature of "adhesion" between the brick and the mortar and did so at a time when interest in the subject was relatively new. Later research has explored and built upon many of these earlier theories, but to this date no consensus exists as to the true mechanism of bond. It is only in fairly recent years that detailed chemical electron microscope and X-ray investigations of the contact zone have been undertaken and yet these reveal little regarding the actual formation processes.

The following literature review discusses many of the presented theories regarding the bond mechanism and the resulting information are considered under various headings.

## 5.1.1 The Theory of Adhesion

Kampf[42] in 1963 in his study of factors affecting bond concluded that bond of conventional mortar and brick is primarily one of mechanical keying rather than a molecular bond. Kampf disputes theories that the bond is formed by one of adhesion alone, since X-ray and thermogravimetric bond investigations have failed to reveal crystalline or amorphous phases. However Kampf does acknowledge that in the case of smooth glass plates, that do exhibit bonding ability to the mortar, there must be the presence of some chemical interaction.

Kampf considers that Portland Cement is a polar compound and may therefore be classified as an adhesive. The polar covalent bonds account for the adhesive and cohesive strength of the mortar. He argues that the strength of such bonds vary inversely with the cube of the distance and therefore argues that any material such as sand, water and air which increase the distance between cement particles, reduce the bonding force.

### 5.1.2 Bonding Layer

Several workers have identified the presence of a definitive layer at the bonding interface, formed by components of the mortar constituent materials. Kampf maintains that in the case of mechanical bond that mortar paste must flow into the surface voids of the brick.

Voss[75] examined micro-photos of thin sections through the mortar joint and was able to observe that where the bond was intimate and continuous, there was a thin layer of material between the brick and the main body of the mortar. He found that this layer appeared to be mainly carbonated lime, which filled all the holes and gaps and penetrated into the voids of the brick. He concluded that the lime was carried into the brick by water as a result of brick suction.

Staley[76] offers further evidence of the existence of a bond layer and concurs with Voss that there is evidence that lime is transferred to the brick interface. However Staley found that for cement rich mortars, which he described as harsher than lime mortars and which lacked "fatness", did not benefit from brick suction; the result was a tentacular contact by fingers of mortar adhering to the brick surface.

Grandet[77] also observed tentacle like contact between the brick and the mortar and was able to show that the bonding layer was in-fact formed by ettringite; sulphate ions from the gypsum, which dissolves from the cement paste are concentrated at the interface forming a layer of ettringite generated by the reation between tricalcium aluminate and dissolved gypsum ($3CaO.Al_2O_3.3CaSO_4.32H_2O$). He concluded that the development of a layer of ettringite at the brick-mortar interface was indicative of good bond. In the case of glass plates, which exhibit no suction, there is no transportation of sulphate ions by water and the contact layer consists mainly of portlandite with only limited formation of ettringite. Portlandite $Ca(OH)_2$ is formed by the hydration of tricalcium silicate.

### 5.1.3 Water Absorption

Most workers are in broad agreement that it is the action of brick background suction which has a contributing effect on the brick-mortar bond. Although there has been a large amount of research undertaken to measure the influence of brick suction, workers have failed to present explanations as to how water transfer contributes to the bond strength. The review of literature in Chapters 6 and 7, discusses in detail the principles of measurement of water transfer and the mechanisms that may be present which in some way contribute to the bond formation process.

Sise, Shrive and Jessop[45] argue that it is the mechanism of water transfer which draws water and associated chemicals into the brick pores and is essential to the formation of good bond.

### 5.1.4 Surface Texture

Thornton[41] examined the influence that surface texture of the brick may have on bond strength. In his study, Thornton chose water penetration as an indicator of bond. He found that the mortar did not penetrate into the voids in the brick surface, regardless of their size or configuration. As a result he found that water could penetrate between the brick surface and the body of the mortar bed. McBurney, Copeland and Brink[40] also carried out permeability tests on small panels of brickwork and determined that differences in surface texture of the brick did not appear to have significant effect on permeability.

Kampf observed that high-suction, wire-cut brick gave higher bond strength than pressed brick of a similar suction rate, which he attributed to better keying of the mortar on the rougher surface of the wire cut brick. However, he also observed that similar bricks of lower suction rate exhibited no significant difference in bond strength; this he explained was due to the fact that bricks of lower suction rate were likely to have been fired at higher temperatures, thus negating the difference in surface texture.

### 5.1.5 Failure Plane

Several workers have observed the phenomena that there would appear to be a difference in bond strength between the mortar bed and the bricks above and below the joint. The work embodied in this research programme recognises that factors leading to the preferential failure of a particular interface could help to indicate the bond formation process or identify parameters which are detrimental to the bond.

In their comprehensive review of literature on brick-mortar bond, Goodwin and West[78] report that several workers have made reference to weaker bond between the brick and the mortar above the bed-joint.

Davison[79] observed that most leaks occurred at the interface between the top of the mortar and the brick above (denoted as top plane failure). Davison attributed this characteristic to lowered plasticity of the mortar bed, caused by water removal due to brick suction from the lower brick, resulting in a reduced bond for the upper brick. Davison made field observations and found that it was not unusual for bricklayers to lay out a considerable length of mortar bed before laying the successive course. Consequently, by the time the upper course was laid the mortar lost considerable plasticity. However, Davison also noted that this mortar bed was laid to considerable thickness and therefore compensated for the effects of the lower brick suction.

Adams and Hobbs[33] observed the position of the joint failure in wallettes and noted that these were predominantly top plane failure. The authors clarified that where frogged bricks were used, the frogs were laid uppermost and therefore provided a greater body of mortar at the upper interface.

Beal[80] reporting on a programme testing the flexural strength of concrete block masonry found that the failure plane was predominately along the lower interface. Beal attributes this phenomena to the fact that little water was absorbed from the mortar by the lower block, prior to placing the top block. The remainder of the mix water flowed under the influence of gravity onto the surface of the lower block, forming a reservoir of water, which he maintained was detrimental to the bond formation.

De Vitis, Page and Lawrence[70] observed the influence of the joint failure position on bond strength tested using a flexural method by the application of a bond wrench. They found that the position of the failure plane was distributed between the upper and lower joint for clay masonry. However for calcium silicate and concrete units they observed a clear trend for failure to occur along the top interface. The authors remain uncertain as to the reason for this trend but offer the explanation that less effective hydration of cementitious products or lack of cement paste at the top interface lead to preferential lower interface bond strength. They argue that gravity plays a role in the transport of water through the plastic mortar joint, possibly resulting in a greater amount of water settling on the lower interface, with a subsequent greater amount of water and paste being absorbed through the lower interface.

Research remains inconclusive regarding the mechanism which promotes a preferential interface failure. One explanation could be that in flexural bending tests, the failure plane is determined by the proximity of the joint closest to the point of maximum bending.

It would appear that there is broad agreement amongst workers that it is the nature of water transfer from the wet mortar bed, between the upper and lower brick which determines the development of the bond. One explanation for the disparity in reported failure planes could therefore be attributed to the suction characteristics of a particular unit. This phenomenon has been observed during this research programme, where for example Fletton clay commons demonstrate almost entirely bottom plane failure, while engineering bricks depict upper plane failure. The mechanisms contributing to bond strength of particular unit suction characteristics are expanded in Chapters 6 and 7.

## 5.2 Methodology for Inhibition of Hydration

The objective of the investigation was to suspend or retard the hydration process and thereby identify the contribution made by the cement to the development of interface bond. The climatic simulation was not intended to test the durability of the brick-mortar interface and therefore did not incorporate wetting of the couplet faces. It was considered that traditional wetting and freezing cycles may lead to disruption of the bond by freeze thaw action, to such an extent that bond strength testing would not have been practicable.

### 5.2.1 Test Regime

The treatment samples were exposed to 6-hours of freezing at -12°C and 20% RH followed by a fast warm-up period of 1-hour. Cycling of freezing as opposed to sustained freezing, was used to represent more realistic weathering patterns of late evening and early morning frost. The samples were then cured for a further 5-hours within the temperature range between 13-25°C. Subsequent freeze cycles were then repeated and each freezing cycle ran for 6-hours with a further 6-hours recovery period.

Two accelerated cycling regimes were carried out using a British Ceramic Research Limited (BCRL) accelerator chamber, each cycle representing a period of freezing and a combination of two simulations aimed to represent 1-day of exposure. Hence 1-day (2 cycles) and 1-week (14 cycles) were used in order to distinguish between the effects of freezing duration. 1-day exposure would determine whether the bond was permanently disrupted by a single isolated process of freezing, while 1-weeks simulation would show any retardation in bond strength compared to normal cured control samples. The conditions inside the test chamber were measured using a Elpro Busch data logger which measured temperature and relative humidity. The recorded data are shown in Figure 5.1. During testing of treatment samples, corresponding control samples were stored under normal curing conditions (20°C, 80% RH) in accordance with other testing conducted for this research programme.

### 5.2.2 Experimental Programme

Bond testing was conducted on samples for both simulations of 1-day and 1-week at specific points along the bond strength development curve. To achieve this, different durations of pre and post-cure were used. Samples were cured normally after manufacture, for a period of pre-cure before being exposed to freeze cycling. Pre-cure age of 1-day refers to samples which were made and exposed to cycling the same day. Generally, samples were made in the morning and presented to the chamber during the afternoon, once the mortar had developed sufficient strength. The first freeze cycle was generally conducted overnight to represent early exposure of newly laid masonry to overnight frost action. On completion of simulation, samples were removed from the BCRL chamber and cured under normal conditions (20°C, 80% RH) for a period of post-cure, prior to bond testing. Table 5.1 demonstrates the array of pre and post-cure cycling.

**Table 5.1: Array of Pre and Post-cure Test Ages**

| | **Post-cure [days] Exposure (1-day)** | | | | **Post-cure [days] Exposure (1-week)** | | | |
|---|---|---|---|---|---|---|---|---|
| **Pre-cure [days]** | **0** | **7** | **28** | **63** | **0** | **7** | **28** | **63** |
| **1** | **2** | 9 | 30 | 65 | 8 | 15 | 36 | 71 |
| **3** | 4 | 11 | 32 | 67 | 10 | 17 | 38 | 73 |
| **14** | 15 | 22 | 43 | 78 | 21 | 28 | 49 | 84 |
| **28** | 29 | 36 | 57 | 92 | 35 | 42 | 63 | 98 |

Each block contains 6 replicates for both treatment and control samples. 6 control couplets were cured normally and tested at same time as corresponding treatment samples.

Each simulation tested 8 combinations of pre and post-cure, with each test age represented by 6 treatment samples and 6 control samples. In total, 48 samples were exposed to any one simulation. Pre-cure samples of 1, 3, 14 and 28 days were tested for 1 and 7 days post cure. The experiment was then repeated for 28 and 63 days post-cure.

Each experiment was repeated twice for the two different cycling regimes, resulting in 4 simulations. In total, 384 couplets were tested for tensile bond following the procedure described in Section 2.2.

The couplets for each treatment were staggered diagonally in a wall, 6 couplets high by 8 couplets wide. This ensured that any localised warm spots within the corners of the chamber would be randomly distributed across the treatment range. Plate 5.1 shows the testing arrangement. The couplets were dry stacked into the chamber and a wall plate was positioned on the top row of couplets and jacked using threaded rods to compress the samples slightly in order to hold them in place during testing. An 80mm thick UPVC backing panel was then positioned on the outside of the accelerator to seal the chamber.

### 5.2.3 Sample Manufacture

All samples were manufactured in accordance with the procedure described in Section 2.2.2. Fletton units were non-suction adjusted. Mortar was batched and mixed following the procedure specified in Section 3.2.2.

The mortar used was a 1:1:5 ½ cement:lime:sand mix with 4ml of air entrainer and 3720ml of water added. The average consistence by dropping ball was measured as 12±1mm. The air content measured by the density method was on average 13.7%.

### 5.2.4 Testing Procedure

Samples were tested once they had reached their specified post-cure age using the procedure outlined in Section 2.2.5. The treatment samples were tested first, followed by the control samples. For any particular age, bond strength testing generally took around 8-hours, leading to a small, but not significant difference in age between treatment and control samples. The failure interface of each sample was recorded as either top or bottom failure.

It was noted that several of the samples had failed during curing and therefore returned a value of zero bond strength. Other samples failed in the test rig under the load of the lower unit and these were also recorded as zero bond strength. A value of zero is considered as a quantitative value in these results, provided that failure occurred during curing and was not attributable to accidental damage. Samples which were notably damaged by handling during manufacture, simulation, curing or testing were recorded as NA and are not featured in the analysis of results.


**Plate 5.1: Showing array of samples presented to BCRL climatic chamber**

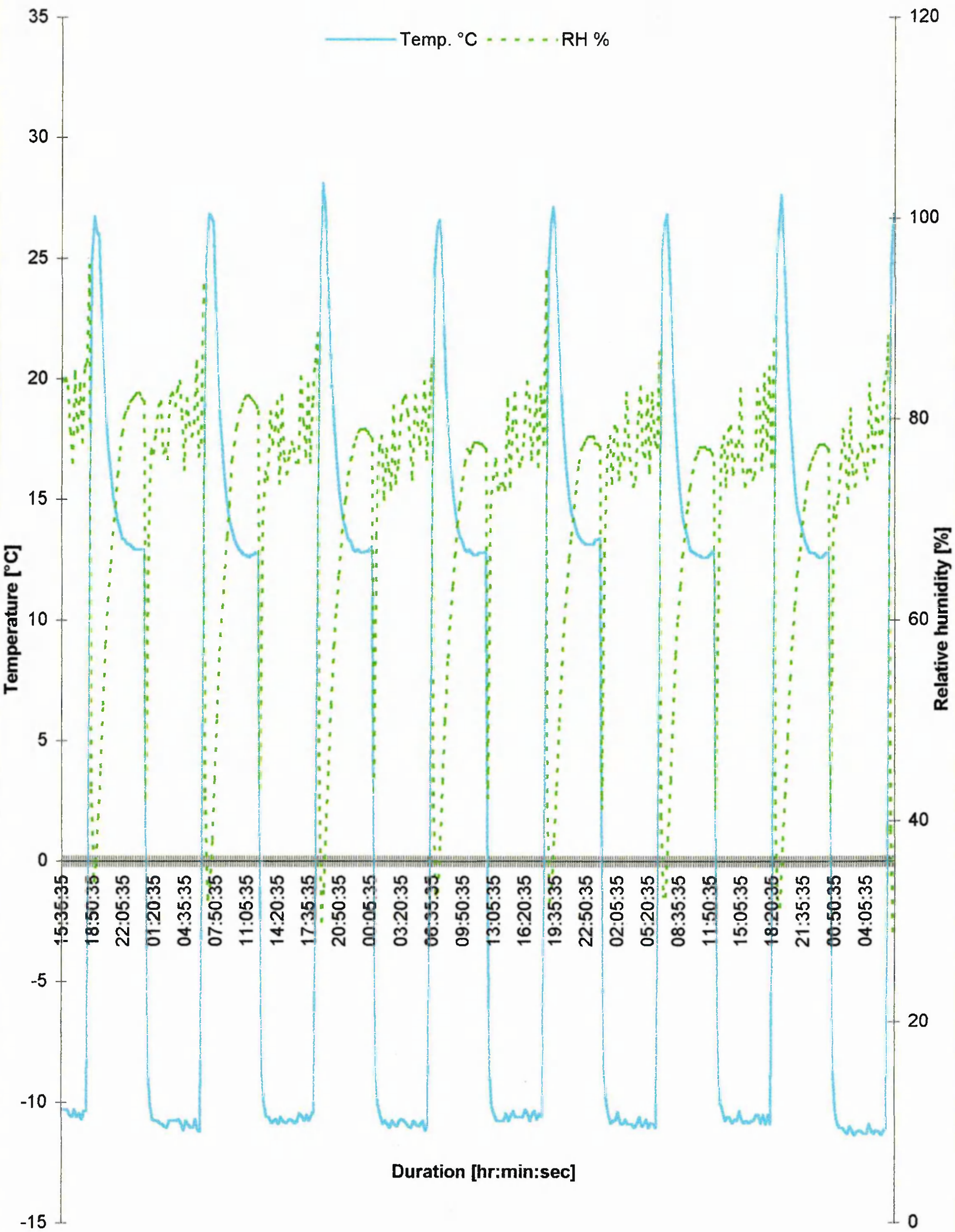



**Fig 5.1: Temperature and Relative Humidity Recorded During Freeze Cycling.**

## 5.3 Experimental Results and Analysis

The original hypothesis predicted a lowering of bond for the treatment samples relative to the normally cured control samples due to the inhibition of the cement hydration process by freezing. Therefore a one-tailed, two sample t-test, which employed a directional hypothesis was used initially for the analysis. However, it was noted that while some treatments showed a significant difference in average bond strengths from control cured samples, the direction of the effect was not necessarily to reduce bond strength. Therefore, a two-tailed, two sample t-test was adopted. A two-tailed t-test is more conservative in its findings than one-tailed test, since the level of significance is halved. Table 5.2 and 5.3 show average bond strength and standard deviation values for both the treatment and control samples exposed to 1-day and 1-week simulation, respectively. Values shown in shaded cells indicate a significant difference at 95% confidence level, between treatment and control sample means. The direction of effect can be obtained by comparing the treatment and control values from the tables. This information is shown in graphical format in Figure 5.2 and 5.3, which shows the average bond strength values for the treatment samples only. Significant difference between treatment and control average bond strength values are marked by horizontal shading.

Analysis of the difference in means for the 1-day and 1-week samples was not made, since there is a difference of six days between treatment ages at test. Consequently, results attributable to duration of freezing cannot be distinguished between because of the difference in the age of the populations.

Figure 5.4 shows the results of all the control sample bond strengths plotted against age. This provides a bond strength development profile between 2-days and 98-days. The solid regression line shows a polynomial curve for data incorporating zero bond strength values. The hatched regression line shows the projection for data values which ignore zero bond strength values. Values of bond strength at 2-years of age, from the same sample population, have been superimposed on Figure 5.4, but have not been used for regression analysis. The two separate data sets at 2-years age show both the bond strength averages and standard deviation, accounting for or ignoring zero values. The samples at 2-years of age demonstrated 40% of bonds failing during curing.

**Table 5.2: Array of Bond Strength Results for 1-Day Exposure Samples and Corresponding Control Samples**

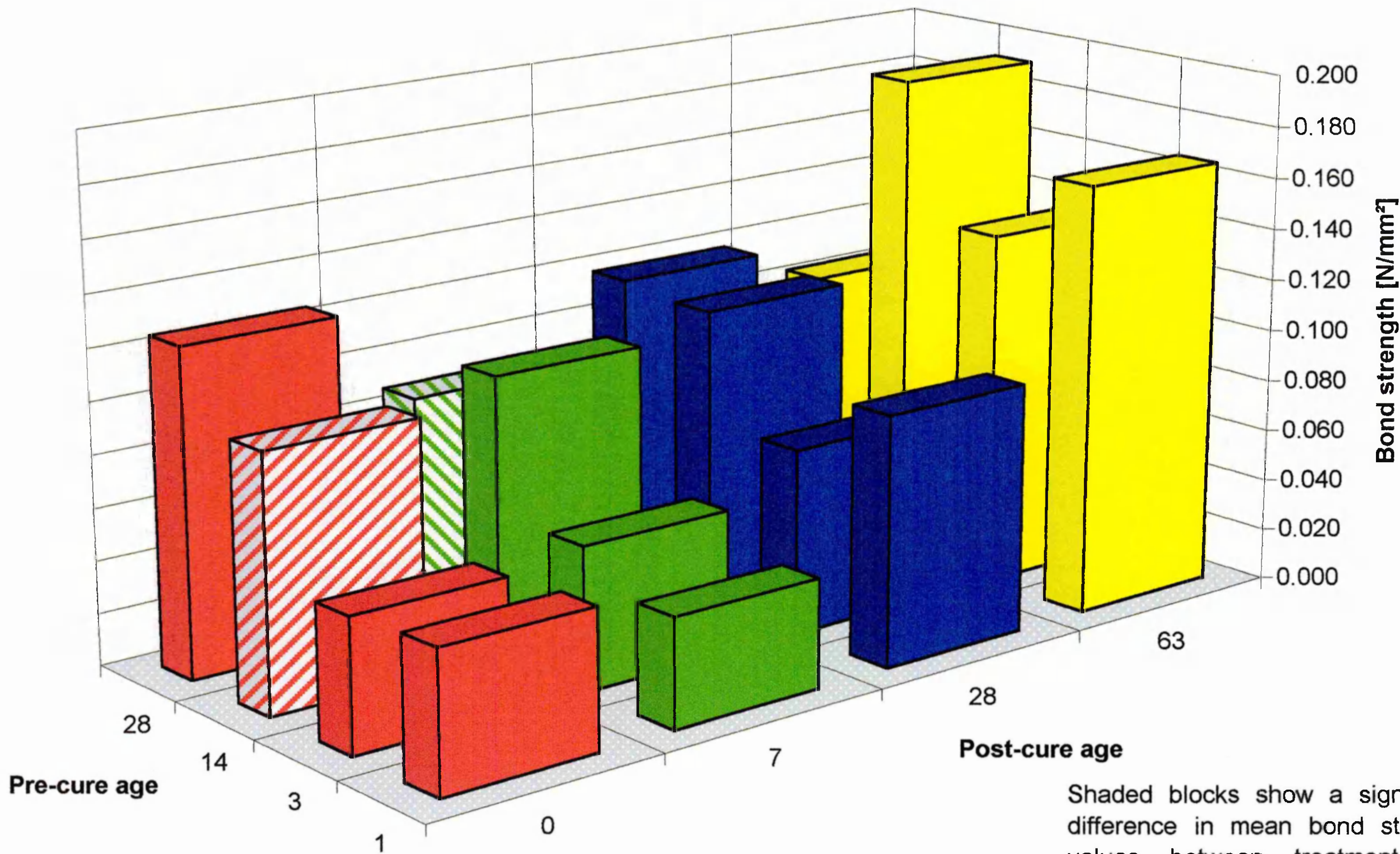
| Bond strength values [N/mm²] | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **Post-cure** [days] | | **0** | | **7** | | **28** | | **63** | |
| **Pre-cure** [days] | | **T** | **C** | **T** | **C** | **T** | **C** | **T** | **C** |
| **1** | Bond | **0.053** | **0.042** | **0.042** | **0.039** | **0.096** | **0.092** | **0.166** | **0.163** |
| | S.dev. | 0.013 | 0.004 | 0.018 | 0.008 | 0.030 | 0.012 | 0.034 | 0.087 |
| **3** | Bond | **0.051** | **0.043** | **0.055** | **0.054** | **0.072** | **0.069** | **0.138** | **0.154** |
| | S.dev. | 0.015 | 0.014 | 0.018 | 0.022 | 0.025 | 0.018 | 0.028 | 0.040 |
| **14** | Bond | **0.098** | **0.066** | **0.107** | **0.092** | **0.115** | **0.130** | **0.191** | **0.161** |
| | S.dev. | 0.025 | 0.013 | 0.034 | 0.023 | 0.017 | 0.045 | 0.037 | 0.047 |
| **28** | Bond | **0.125** | **0.104** | **0.088** | **0.098** | **0.118** | **0.099** | **0.103** | **0.169** |
| | S.dev. | 0.040 | 0.056 | 0.016 | 0.053 | 0.015 | 0.056 | 0.083 | 0.044 |

T= treatment; C= control. Bond strength values taken for 6 replicates per treatment. Shaded cells show a significant difference between the means of the treatment and control samples at 95% confidence level. Zero values of bond strength have been incorporated in the analysis.

**Table 5.3: Array of Bond Strength Results for 1-Week Exposure Samples and Corresponding Control Samples**

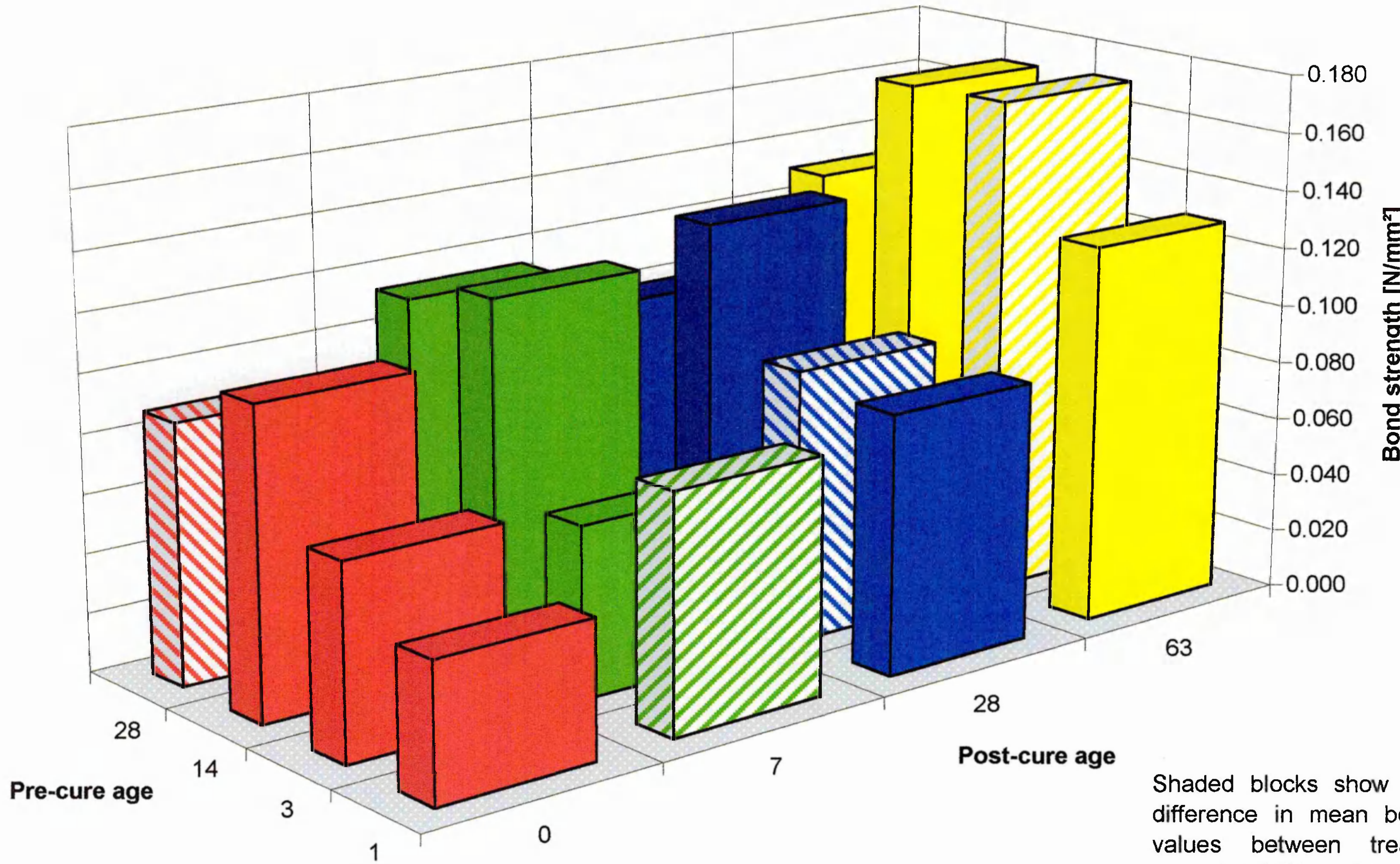
| Bond strength values [N/mm²] | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Post-cure [days] | | 0 | | 7 | | 28 | | 63 | |
| Pre-cure [days] | | T | C | T | C | T | C | T | C |
| 1 | Bond | 0.046 | 0.041 | 0.080 | 0.056 | 0.088 | 0.117 | 0.129 | 0.136 |
| | S.dev. | 0.008 | 0.013 | 0.017 | 0.018 | 0.036 | 0.034 | 0.049 | 0.032 |
| 3 | Bond | 0.065 | 0.050 | 0.058 | 0.079 | 0.093 | 0.149 | 0.171 | 0.124 |
| | S.dev. | 0.021 | 0.014 | 0.012 | 0.026 | 0.023 | 0.028 | 0.017 | 0.027 |
| 14 | Bond | 0.104 | 0.097 | 0.123 | 0.105 | 0.112 | 0.140 | 0.170 | 0.121 |
| | S.dev. | 0.019 | 0.005 | 0.022 | 0.017 | 0.072 | 0.016 | 0.052 | 0.065 |
| 28 | Bond | 0.088 | 0.131 | 0.095 | 0.110 | 0.082 | 0.104 | 0.087 | 0.113 |
| | S.dev. | 0.017 | 0.024 | 0.053 | 0.012 | 0.046 | 0.031 | 0.084 | 0.030 |

T= treatment; C= control. Bond strength values taken for 6 replicates per treatment. Shaded cells show a significant difference between the means of the treatment and control samples at 95% confidence level. Zero values of bond strength have been incorporated in the analysis.




Shaded blocks show a significant difference in mean bond strength values between treatment and control at 95% confidence.

**Fig 5.2: Bond Strength Values Exposed to 1 days Simulated Freezing at a Given Pre-cure Age and Tested at a Specified Post-cure Age.**




Shaded blocks show a significant difference in mean bond strength values between treatment and control at 95% confidence.

**Fig 5.3: Bond Strength Values Exposed to 1 Weeks Simulated Freezing at a Given Pre-cure Age and Tested at a Specified Post-cure Age.**

## 5.4 Discussion of Results

Comparison between the mean bond strength of the treatment samples with the control samples, reveals that the differences are not as marked as originally anticipated. This would indicate that either suspension of hydration has no significant influence upon the development of bond or that duration of inhibition by freezing was not sufficiently prolonged. It is probable that samples tested at an early age have not developed sufficient bond strength for the treatment effect to be detected.

In general, treatment samples of either an early pre-cure or post-cure age depict a greater bond strength compared to their corresponding control samples. Treatment samples of more developed pre or post-cure age demonstrated the reverse effect, with a reduction in bond compared to the control samples. This trend may be demonstrated by examining Tables 5.2 and 5.3 as matrices. The cells with the darkened border represent reduction in bond strength for the treatment samples compared to the control. A hypothetical diagonal line running through these cells shows that those combinations of pre and post-cure ages lying to the right of this line generally represent samples of total age of 35-days and above. This could indicate that inhibition of hydration has an advantageous effect upon bond strength development for early age samples, but that the same retardation process applied to samples aged 35 days and over may potentially be disruptive.

The results fail to show a trend that the treatment and control samples are significantly different from each other. As discussed in Section 5.3, a one-tailed t-test may have produced more pairs of data showing significant differences in their mean bond strength. However, in order to apply such a directional hypothesis, there first must be evidence to show that there is a specific effect of treatment. In this experiment, the directional effect of treatment by freezing upon tensile bond strength remains unproven and there has been no precedence established by other research.

Notwithstanding, despite individual paired sets of data not showing significant difference in their means, collectively the array may show a certain characteristic which may indicate a weak effect.

The 1-day and 1-week exposure cycles shown in Figures 5.2 and 5.3 show a general increase in bond strength development as the sample age increases. Samples with pre-cure age of 1, 3 and 14 days for each of the exposure cycles show a marked increase in bond strength with increasing post-cure age. However, samples cured for 28-days before exposure demonstrate a levelling-off of bond strength development with increasing post-cure age.

The results for samples exposed to 1-day of freezing, shown in Figure 5.2, demonstrate that no permanent impairment to bond strength development was initiated by the freezing process. Samples exposed to 2-cycles of freezing the same day as manufacture, demonstrate an early bond strength development with continued strength gain with increasing post-cure age.

All incidences of bonds failing during curing occur in samples aged 28-days or greater. This could indicate that the bond formation mechanism may still continue past the accepted 28-day benchmark for testing.

Figure 5.4 illustrates that the secondary phase of the bond strength development curve maximises between 60-days and 85-days. This plateau is characterised by an increase in occurrence of zero bond failures. Consequently the regression lines diverge at this point; with those values which take no account of zero bond projecting a continued increase in bond strength development; the regression line which incorporates the zero bond strength values show an overall decline in bond strength post 70-days.

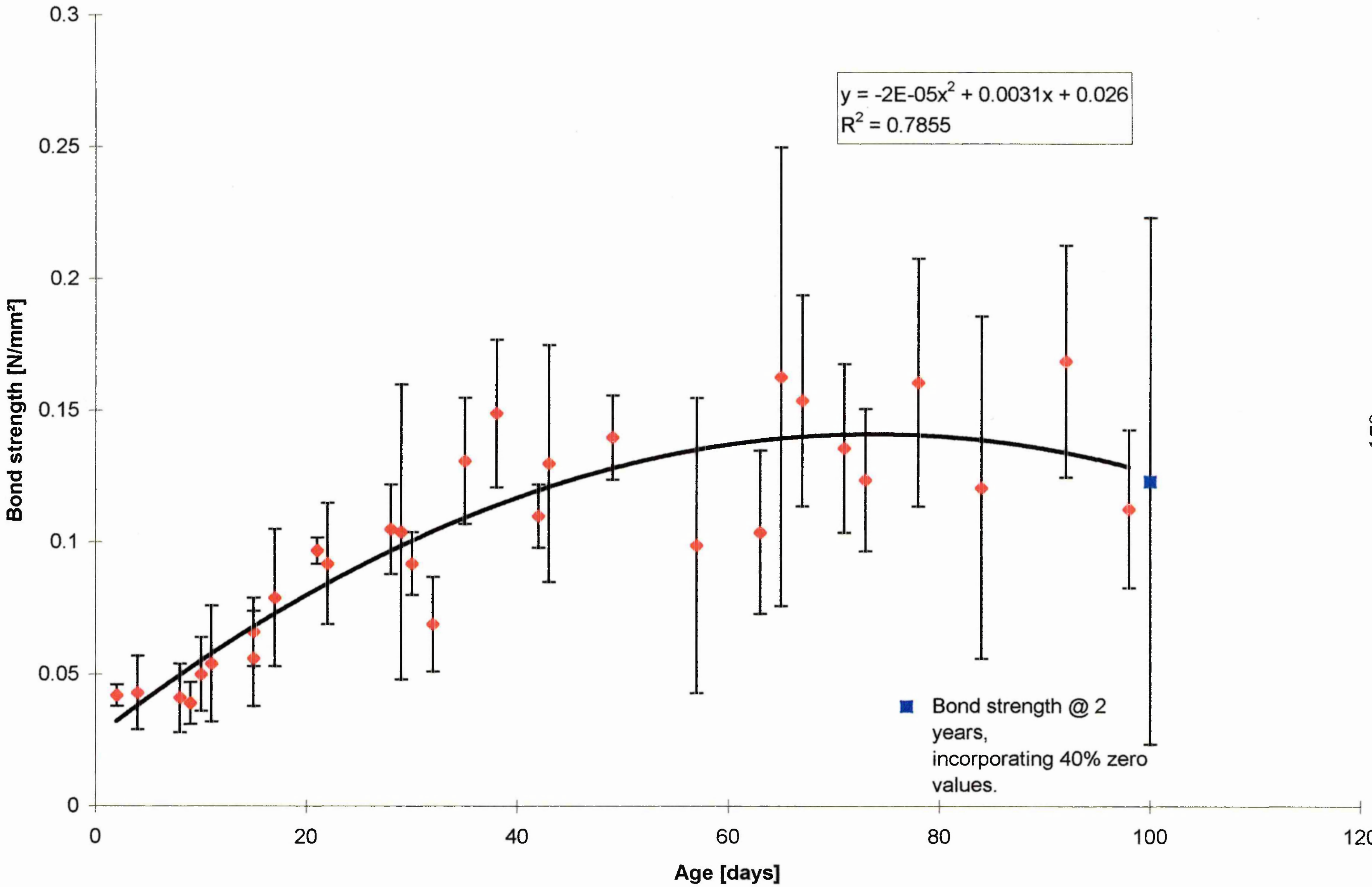



Fig 5.4: Bond Strength Development Curve From 2 Days to 14 Weeks

## 5.5 Concluding Remarks

The effects of inhibition of hydration by freezing upon bond strength development are inconclusive. The early tensile bond development discussed in Chapter 4 highlighted that the formative bond could be impaired if the mechanism were disrupted. Exposure to early freezing was considered to be the most likely influence to formative bond disruption of newly laid masonry. This experimentation demonstrates that there is no such impairment to the bond formation by exposure to sub-zero temperatures, provided that the effects of freeze-thaw action do not disrupt the bond mechanically.

The above trend was also reported by Baker[34] who found that higher flexural bond strengths were achieved for wallettes which were initially cured at 0°C, compared to those cured at 20°C. He found that repeated freeze-thaw cycles eventually led to a reduction in flexural bond strength. He concluded that while compressive strength of brickwork is adversely affected by freezing, bond strength remains unaffected.

Indeed, there is tentative evidence to suggest that retardation of the hydration process at an early age may actually be advantageous to the bond development. It is considered that freezing may control the rate of shrinkage in the mortar by suspension of the free water. At ages 35-days post manufacture, once cement hydration is established, the direction of the effect of freezing may be less pronounced.

The 28-day bond strength measurements for predicting long-term bond strength performance of masonry, must be regarded with caution. The bond strength maturity curve shown in Figure 5.4 demonstrates that an overall reduction in bond strength post 28-days can occur, depending upon how the results are interpreted.

Inclusion of the zero bond strength values in calculation of the mean is controversial. If zero values are excluded, bond strength maturity curves demonstrate sustained increase, as experienced by previous investigations described in the literature review in Chapter 4. However, at 2-years of age, couplets show up to 40% zero failure in the lower plane and this result is so significant that zero values cannot be overlooked. It is suggested that occurrence of zero bond strength should be included in treatment averages, since zero values clearly occur due to the ongoing bonding mechanism.

The occurrence of such a high proportion of zero bond strengths over time may lead to the perception that continued shrinkage of the mortar bed may well result in the eventual rupture of all brick-mortar bonds over time, resulting in brick walls simply taking the form of a vertical pile of units, with no tensile or flexural bending strength. However, not all samples exhibit such failure and secondly the force of pre-compression from courses above will influence the frictional bond at the interface.

Continued drying shrinkage does cause failure in some isolated samples and calls for further investigation of the parameters which may influence this apparent incompatibility between the brick and the mortar.

Early plastic shrinkage and consequently longer-term drying shrinkage are considered to be synchronous with unit water absorption characteristics. A detailed study of the nature of brick water absorption characteristics is required to enable shrinkage parameters to be identified.

## References

16. British Standards Institution. Code of Practice for the Use of Masonry: Part 1: Structural Use of Unreinforced Masonry. BS5268: Part 1:1992.

33. Adams, M.A., Hobbs,D.W. Bond Strength of Brickwork From Couplet and Wallette Tests: Some Comparative Results. Masonry International Vol. 8, No 1, 1994.

34. Baker, L.R. Some Factors Affecting the Bond Strength of Brickwork. 5th International Brick Masonry Conference, session 2, paper 9,1979.

35. Isberner, A.W. Properties of Masonry Cement Mortars. Designing Engineering and Constructing With Masonry Products. Gulf Publishing Company, Houston, Texas 1969.

41. Thornton, J.C. Relation between Bond and the Surface Physics of Masonry Units. Journal American Ceramic Society. 36, (4), 105, 1953.

42. Kampf, L Factors Affecting The Bond of Mortar to Brick. ASTM Special Technical Publication No 320 p67 ;Symposium on Masonry Testing, 1963.

45. Sise, A., Shrive, N.G., Jessop E.L. Flexural Bond Strength of Masonry Stack Prisms. Proceedings of the British Masonry Society, No 2 pp 103-107. April 1988.

70. De Vitis, N., Page A.W., Lawrence S.J. Influence of Age on Masonry Bond Strength: A Preliminary Study. Masonry International Vol. 12, No.2, 1998.

75. Voss, W.C. Permeability of Brick Masonry Walls: a Hypothesis. Proceedings ASTM 33, 670, 1933.

76. Staley, H.R. A Petrographic Study of the Bond Between Bricks and Mortar. Trans. British Ceramic Society. 39, 85, 1940.

77 Grandet, J. Contribution A l'etude de la liasison entre la pate de ciment Portland et les terres cuites. Terre Cuite (51), 1971.

78. Goodwin, J.F., West, H.W.H. A Review of Literature on Brick Mortar Bond. British Ceramic Research Association adapted from technical Note 308.

79. Davidson, J.L. Loss of Moisture From Fresh Mortars to Bricks. Materials Research Standards, part 1, p385. 1961.

80. Beal, D.L. Concrete Block Masonry Bond Strength. Proceedings of the 7th International Brick Masonry Conference, Melbourne, Australia, Feb. 1985.

# SURVEY OF UNIT SUCTION CHARACTERISTICS

## Chapter Summary

*Chapter 6 investigates the relationship between conventionally measured brick absorption characteristics and direct tensile bond strength of brick-mortar interfaces.*

*The Initial Rate of Suction Test described in BS 3921*[66] *has been adopted as a criterion*
*The Initial Rate of Suction Test described in BS 3921*[66] *has been adopted as a criterion*
*to determine specific bands of unit suction rate within a population of fletton bricks obtained from the same kiln batch.*

*Couplets were manufactured using bricks of similar suction rate and a 1:1:6 cement: lime: sand mortar as described in Chapter 3.*

*An experimental program was designed to test the effect of both differing levels of suction rate and the influence of suction adjustment of units by docking, upon the measured value of direct tensile bond strength.*

# 6.0 Introduction

The phenomenon of water transfer through porous materials is of particular importance to the study of masonry, in two respects. Firstly the presence of water and its passage within the material influences the durability of the finished brickwork and concerns matters such as rising damp, frost damage, interstitial condensation, sulphate attack, persistent efflorescence and rain penetration. Secondly, the uptake of water from the fresh mortar bed due to brick background suction and its associated transfer across the brick-mortar interface during the initial bonding phase, are considered to contribute significantly to the development of the interface bond.

The work discussed in Chapters 6 and 7 is concerned primarily with the mechanism by which water is removed from the wet mortar bed by brick background suction

The mechanism of water transfer through the finished brickwork holds no relationship to the processes that contribute to the initial development of the interface bond, however studies of water transport mechanisms through porous media help to create an understanding of the activities involved.

The mechanism by which this water transfer influences the formation of the interface bond remains conjectural. The presence of moisture in the mortar bed and the nature and quantity of its subsequent removal may influence a number of factors which contribute simultaneously to the formation of bond. Likewise, background processes at work may mask determination of specific bonding parameters.

For example, the degree of cement hydration and the creation of hydration products such as ettringite, together with the redistribution of particulates within the mortar bed and across the brick-mortar interface, remain indistinguishable from the influence of water transfer alone and may contribute to, or hinder the formation and performance of the bond.

Removal of the mortar mix water from the fresh mortar bed into the body of the brick leads to volumetric shrinkage of the mortar and subsequent moisture expansion of the brick. This simultaneous lateral differential movement at the bonding interface must have repercussions on the ability not only to form bond but also the time-scale of the bond formation period. The presence of water drawn into the interface layers of the brick bedface may subsequently contribute further to the bond development process by providing a reservoir of water available to satisfy the curing environment of the bond layer. This work recognises that due to the processes described above, the bond formation at the lower interface of the joint may potentially be different to that occurring at the upper interface of the brick-mortar joint.

The level of water uptake by a masonry unit is generally measured by the Initial Rate of Suction (IRS) test outlined in appendix H of BS 3921[66]. This test measures the quantity of water absorbed by a unit area of bedface, submerged to a depth of 3-mm, over a 60-second period ($kg/m^2.min$). The Initial Rate of Suction is sometimes referred to as the Initial Rate of Absorption test.

Despite being widely accepted as a means of measuring the water uptake characteristics of a particular masonry unit, the terminology used to describe this test substantiates that the nature of water uptake may not be solely restricted to the quantity of water removal, but may also be a function of the rate and force of water transfer. For example, the term "absorption" tends to be associated with a quantity of water removed. Measurement of the mass of water absorbed over a period of 1-minute from a free water surface does not reflect necessarily the quantity of water removed from a fresh mortar bed over the bond formation period.

The "rate" of water uptake can similarly be influenced by the retentive behaviour of the mortar. The speed of water transfer may not necessarily be associated with the quantity of water removal. The velocity of water flow on the other hand, has alliance with the term "suction", which is a measurement of force, rather than of quantity and could contribute to the redistribution of fine materials within the mortar bed.

A further consideration is the behaviour of the mix water. Admixtures such as air entrainers, retarders, plasticisers and lime in solution, will all influence the surface tension properties and viscosity of the water and hence the level and rate of water uptake.

The work discussed in Chapter 3 reports that for a mortar mix to attain a suitable level of workability, the volume of mix water should roughly match the volume of cementitious material. It has previously been reported in Chapter 3 that approximately one-third of the initial mix water is required for cement hydration. If the quantity of mix water removed by brick background suction is too low, then a high water-cement ratio and large volumetric shrinkage will occur within the mortar bed. Alternatively, if the level of water uptake is excessive, then the extent of cement hydration will be impeded.

The absorption characteristics of a particular type of brick, may in turn have significant impact on the productivity of bricklaying process. If the initial rate of absorption is too "harsh", then the time period in which the bricklayer can adjust the course to line and level will be dramatically reduced. Plate 6.1 demonstrates how quickly the mortar can stiffen, when subjected to brick suction forces. On the other hand, if there is too little brick suction, as for example engineering bricks, units may "float" on the wet mortar bed, increasing the likelihood for the wall to become out of plumb and restricting the productive height of subsequent courses.

According to the literature, the problem of excessive absorption of masonry units has been recognised for at least 270 years[81] [82] [83] [84] and has traditionally been compensated for on site by suction adjustment of the units prior to laying. In practical terms this means the "docking" or "dunking" of the units, in water, by the operator, to compensate the units absorption characteristics. This process is typically subjective and is controlled by the bricklayer's own assessment of the particular unit characteristics, the workability of the mortar and often weather conditions on site. Consequently, suction adjustment is rarely carried out systematically and may vary between sites and operatives.

To this end, suction adjustment of masonry units at the brick laying location, may induce more inherent variability in the bond development process, than modification of the mortar workability properties on site. Although the practice of suction rate adjustment is a widely adopted site practice, there is very little evidence in the research to suggest that the effect upon the structural performance of the bond strength has been investigated in detail.

The only available recommendation to designers is mentioned in BS5628:Part 3: Section 32.3[67], which states that "in fired-clay brickwork, adjustment of the suction rate of bricks at the time of laying may be required by the designer for structural reasons. The consistency of the mortar should be adjusted or bricks should be wetted (docked) for not longer than 2-minutes just before use". However, there is no evidence to suggest that the above recommendation has been supported by experimental work

The presence of perforations or frogs, the processes of extrusion or pressing, characteristics of the clay, firing temperature and regime, together with the method of stacking and location within the kiln, all account for inherent variability in brick absorption characteristics. Clearly the variability of unit water absorption characteristics cannot be controlled at the point of manufacture and therefore a test is required which will be sufficiently sensitive enough to detect what effect such characteristics might have upon the water abstraction from the mortar.

Adopting established methods of measuring brick absorption levels by initial suction rate tests, this work attempts to address the extent to which the development and performance of the interface bond is influenced by brick background suction. A two factor experiment was designed to investigate both the influence of initial suction rate upon tensile bond strength and the relevance of suction adjustment of the units prior to laying.

The need to detect distinct changes in bond strength due to the effects of specific treatments calls for the use of statistical tests, which are capable of determining discrete differences between the mean bond strengths for any treatment sample population. To be confident that tests yield reliable results, experimental design must cater for large sample replication. As discussed previously in Chapter 2, the direct form of bond test adopted throughout this research programme facilitates the consistent production and testing of a large sample population, which may not readily be achieved using some flexural forms of testing.

**Plate 6.1a** **Showing that the initial bond is immediately capable of supporting the weight of the lower brick**

**Plate 6.1b** **Showing the effect of unit suction upon the fresh mortar slab after the bond is broken**

## 6.1 Discussion of Literature Relating to Unit Suction Rates

It remains generally accepted amongst researchers that water transfer from the fresh mortar bed, encouraged by the natural capillary suction background of the brick bedface, contributes in some way to the development of the interface bond. In turn, the degree of water removed from the mortar bed in its wet state can be related to the subsequent properties of the hardened mortar. Research workers have attempted to measure and characterise some of the parameters of water transfer within porous materials, in an attempt to develop an understanding of the processes at work at the brick-mortar interface which contribute to the bond strength.

Traditionally, elementary assessments of a materials ability to absorb water have been obtained through the measurement of solid and bulk density by vacuum saturation, boiling or 24–hour submersion. Appendix E of BS 3921[66] outlines the method of determining water absorption by 24-hour cold immersion.

Anderegg[85] measured the absorption of brick bed-faces immersed in 3-mm of water for 10-minutes and recommended that for summer work, the absorption should be in the range of 1% to 3% of the brick weight and for winter work 3% to 5%. Voss[75] suggested an absorption of between 5% and 10% of the brick weight, during a 48-hour immersion, as being the most satisfactory for the formation of a bond layer.

Sise, Shrive and Jessop[45] maintain that of unit properties, the saturation coefficient, and 24–hour absorption were found to affect bond strength most significantly. The saturation coefficient is the measure of how much water a unit can absorb within 24-hours, compared to its total capacity obtained by 5-hour boiling.

The authors argue that since the bond takes some time to develop, it is reasonable that 24-hour absorption of moisture by the unit over that time affects the bond strength. This does not mean to say that the initial rate of suction is not important, just that saturation is a more reliable measure. They argue that while modulus of rupture obtained by flexural bending calculations may have significant results, such factors may be due to other parameters such as porosity and pore size distribution of the unit.

The authors indicated a moisture content of 65-70% of 24-hour absorption to promote optimum bond strength development. They conclude that at lower initial values of moisture content, too much water is drawn away from the mortar for sufficient hydration of cement. When the water content is too high, no water and associated chemicals will be drawn into the pores of the unit and this latter process may be essential to formation of good bond. Finally the authors suggest that unit properties have more influence on bond strength than mortar properties.

Gummerson, Hall and Hoff[86] in their study of water transport in masonry structures conducted in 1980, emphasise that water absorption porosity only measures the water holding capacity of the porous material and fails to define the way in which the water moves and is retained by such a materials. They remark that saturation tests define a material under the unusual conditions of saturated flow, which are rarely achieved in practice.

The Initial Rate of Suction Test is considered by many workers to reflect more closely the degree of water removal by capillary forces from the brick bedface and is now the usually accepted standard measure of brick suction. It is expressed as the weight of water absorbed over a period of 60-seconds by the brick bedface area, when immersed to a depth of 3-mm. In the UK, the value is expressed as $kg/m^2.min$ and is sometimes referred to as the Initial Absorption Rate. Workers have attempted to correlate tensile bond strength of brick-mortar joints with initial suction, obtained by this measurement.

Morgan[87] stated that in order to achieve optimum brick-mortar bond strength with a particular brick type, there must be compatibility between the absorptive properties of the brick and the water retentive characteristics of the mortar. Morgan investigated what he termed the "peculiar" bonding characteristics of some types of bricks, particularly calcium silicate and concrete bricks. The test procedure he adopted was designed to determine the amount of water absorbed through a bedface of the brick for each of a number of immersion times, between 1-minute and 24-hours, using the standard suction rate test. Morgan determined that the relationship of the amount of water absorbed with time, followed a cube-root relationship. This phenomena is also reported by other workers[88] [89] [90]. From this relationship, Morgan was able to establish the instantaneous absorption rate from the first derivative (with respect to time), of the fitted lines. He concluded that the water absorption characteristics of the calcium silicate and concrete bricks, with low early absorption rates coupled with relatively high long-term absorption rates, despite reflecting medium to high IRS values, contrast sharply with the characteristics of the clay bricks investigated. Morgan associated these trends with the significant source of problems regarding the development of a satisfactory bond strength of calcium-silicate and concrete bricks and argued that the IRS test may not be the reliable indicator of desirable mortar properties required to reach compatibility with concrete or calcium silicate bricks.

De Vitis, Page and Lawrence[70] also observed significant differences in the rate of absorption for various unit types. The clay and concrete units exhibited similar trends, but demonstrated very different levels of absorption. The calcium silicate unit depicted low absorption values after 1-minute, (as measured by the IRS test), but showed extremely high values after 7-days. The authors concluded that the 1-minute absorption value (IRS) is not necessarily the best indicator of unit water demand.

Anderegg[91] investigated the abstraction of moisture from lime mortar by bricks taken from different parts of the kiln. He also correlated the bond and compressive strengths for 1:2:9 mortar cast between several types of bricks. He noted the very rapid initial rise of water in the dry-press bricks, indicating the presence of relatively large pores. Conversely, the very slow absorption of the hard-burned bricks might be associated with finer pores. Anderegg observed that the brick with the highest initial rate of absorption, removed the smallest quantity of moisture from the lime mortar. In addition, amongst the most absorptive bricks, most moisture was removed by some of the bricks having the lowest initial absorption. Anderegg explained this apparent anomaly by suggesting that the higher the initial absorption rate, the greater was the tendency to form a congealed layer at the surface of the mortar. He maintains that moisture is apparently removed so rapidly from the surface of the mortar that a condensed layer of constituents is formed having a permeability varying inversely with the initial suction. Consequently, a steep moisture gradient is formed between the surface layer and the interior of the mortar. In spite of the presence of this congealed layer, Anderegg noted that the suction force would continue to abstract moisture over a period of time.

For most of the bricks used in Anderegg's experiment, it was noted that the bond strengths increased with brick absorption, reaching an optimum strength, followed by a decline in bond strength. Anderegg argues that the increase in bond strength was due to a reduction in the water-cement ratio, while the further decline resulted from the congealed layer offering poorer contact. Although Anderegg fails in his paper to report upon the method of bond testing adopted, he does note that the compressive strength of the mortar also increased with the absorption rate. The observation that there existed a relationship between bond strength values and compressive strength values with increasing levels of water absorption, suggests that a flexural form of bond testing was utilised. As discussed previously in Chapter 2, flexural forms of testing may reflect more specifically properties of compressive strength, rather than those of direct tensile bond.

Voss[75] examined microphotographs of thin sections through mortar joints and observed that where bond between brick and mortar was intimate and continuous, there appeared a thin layer of material between the brick and the main body of the mortar. This layer, which was mainly carbonated material from lime, filled all the holes and gaps between the brick and the mortar and penetrated small openings in the brick. Voss concluded that lime was carried to the brick face in water drawn by brick suction, prior to the hardening of the mortar. These observations led him to the further conclusion that this action forms the bond layer at the brick-mortar interface.

Davison[79] reports that bond strength and resistance to rain penetration are closely interrelated. He argues that the plasticity of the mortar determines its ability to flow and spread and hence determines the intimacy of contact between the brick and the mortar. The subsequent plasticity of the mortar influenced by brick suction characteristics, may influence both leakage and bond strength performance. Conducting leakage tests on small panels of brickwork at the Division of Building Research of the National Research Council of Canada in 1961, Davison confirmed observations of previous workers[92] [93] that un-bonded areas between brick and mortar commonly form a route for penetration of rain. He discovered that most leaks occurred at the interface between the top of the mortar bed and the bottom of the brick. Using flexural bond strength tests, he determined that most fractures occurred in the top plane. The leakage path, outlined by staining from impurities in the water, was observed on the top of the mortar bed. Davison explains that the reason for this difference in bond strength between the brick above and the brick below the mortar joint, could be due to the reduction in plasticity of the mortar beds upper surface being reduced by the prolonged contact with the brick below during, the laying process.

Davison concluded that the loss of a relatively small percentage of the total moisture content can result in a significant drop in the flow of a mortar and also in the plasticity of that mortar. Davison determined that the greatest moisture losses occurred from the masonry cement mortar, which have the lowest retentivity; while the smallest losses were from the 1:2:9 mortar, which had the highest retentivity.

Davison reported that conclusions reached concerning the bonding life of the mortar bed after its contact with the brick are based upon the assumption that moisture losses occur equally from all parts of the mortar bed. It is possible however, that there is a moisture gradient resulting from maximum losses at the mortar surface in contact with the brick, and that the amount of moisture lost decreases as the distance from the interface increases. Visual examination of specimens removed from the mortar beds during moisture content determinations revealed that mortar at or near the contact surface was dry and stiff in appearance in comparison with the wetter and more plastic mortar within the depth of the mortar bed. The results clearly establish the presence of a moisture gradient in the mortar bed.

Davison's findings suggest that the mechanical properties of the mortar bed in the direction of depth may vary. Consequently, the preferential failure plane of masonry couplets tested in flexural bending may be influenced by these findings.

Davison's observations that most joint fractures occurred in the upper interface, contradicts the results obtained for most of the research work presented in this thesis, in which the majority of fractures depict bottom plane failure. The discrepancy in this case, due to the determination of the presence of a moisture gradient, cannot simply be explained by the difference in bond testing methods alone. Unfortunately, Davison made no similar observations for mortars in contact with a top brick, and therefore no allowance for the influence of gravity upon this moisture gradient between the upper and lower interfaces can be made.

Davison found that the moisture loss curves were similar for mortars in contact with different types of bricks having similar IRS values. He also determined that inferior bond strengths were attained from panels containing bricks with higher initial rates of absorption. The results, which indicated similar moisture loss patterns from the mortars cast against bricks of similar IRS, regardless of differences in manufacture, would suggest that the IRS test here, was a reliable predictor of brick-mortar water transfer parameters.

Murray[89], who utilised the IRS test to study the bonding of renders, reported that the standard suction rate test over 1-minute was helpful in assessing the effect on the immediate working characteristics of a mortar between bricks. However, he concluded that significant absorption continued beyond this time and that long term absorption rates could readily be obtained by an extension of the IRS test. He argued that for bricks, it can be shown that the volume of water absorbed through a unit area varies linearly with the square root of time. Hence a constant, the sorptivity factor (S), can be obtained as the gradient of the straight line obtained when the volume of water absorbed (I), is plotted against the square-root of time (t); giving the expression $I=St^{1/2}$. The author found that higher adhesion values were obtained for renders on dry bricks with greater sorptivity. However, wet bricks have low sorptivity and consequently a poor bond. It is noted that these tests were conducted on the stretcher face of the brick. The description of sorptivity has also been reported by Gummerson, Hall and Hoff[86] and will be reported in further detail in Chapter 7.

Gummerson, Hall and Hoff report how the extraction of water from fresh mortar depends upon the water content and hydraulic properties of the masonry material. They measured weight loss of mortar pats of 1:¼:3 placed on brick and an autoclaved aerated concrete (aac) block. The results indicated that in the case of a particular brick material, for docking in water to have a significant influence, the time of docking must be prolonged. In contrast, the results for the aac block show that relatively short times of wetting result in a significant reduction in the amount of water withdrawn. Examining the fundamental hydraulic parameters they suggested that the brick has more sorptivity than the block and will therefore absorb much more water during the wetting process. The brick also has a higher diffusivity and therefore the water absorbed into the surface layers is rapidly redistributed into the drier regions of the brick; suction to the mortar will not be significantly affected until a substantial amount of water has been removed. The aac material absorbs less water but has a much lower diffusivity, consequently the water absorbed into the surface layers will be much less rapidly redistributed. The surface will therefore remain wet for a longer period and affect the suction accordingly.

The authors state that short term docking of relatively absorbent clay commons will not affect the amount of water withdrawn from the mortar over, say, 30-minutes, although it may help initially with the laying process.

There are further examples in the literature which refer to wetting of highly absorptive bricks. In their review of literature on brick-mortar bond, Goodwin and West[78] reported that Collins[94], Palmer and Hall[95], Fishburn[96], Anderegg[91] and Forkner et al[97] all found that the strength of the bond was increased for highly absorptive bricks if wetted before laying. Goodwin and West suggest that if too much water is absorbed by the brick surface, the bricks may float on the wet mortar bed during the laying process and bond strength will be impaired.

A practical field test to determine the need for wetting brick was described by Houston and Grimm[98]. They determined that saturating brick reduced the bond strength.

Kampf[42] held reservations about the benefit of pre-wetting on site. His objections were that it is seldom done, when done it is insufficient and wetting is never uniform.

Spradlin[99] in the United States alleges that bricklayer productivity is reduced 23% by the use of "vitrified face brick", however this was subsequently refuted by Grimm and Fowler[100] in 1985.

Kampf refers to workmanship and maintains that realignment of bricks after the mortar begins to stiffen will destroy the bond. The time over which bricks can be realigned without the bond being weakened is greatest for low suction bricks and high water retentive mortars.

Haller[101] noted that bricks with a high initial suction rate tended to result in slender walls of reduced strength, presumably on grounds of intolerance of the mortar to adjustment of line and level.

Before suggesting the Initial Rate of Suction test as a yardstick for predicting moisture content losses, Davison[79] thought it desirable to investigate the moisture loss from mortars in contact with different bricks having similar values of IRS. These combinations might result in bricks having identical IRS values but different absorption patterns beyond the first minute.

Davison found that moisture loss curves were all quite similar for mortars in contact with different bricks having similar IRS values. Results indicate similar moisture loss patterns from mortars to bricks with similar IRS values, regardless of differences in manufacture.

Collin[94] investigated the tensile and shear strength of various assemblies constructed with seven types of brick and three types of mortar and arrived at the following conclusions:-

1. Absorption characteristics have a definite relationship to the bond strengths developed with various mortar uses.

2. Low absorption bricks develop medium bond strength with both cement and cement lime mortars, when set either dry or wet.

3. Medium absorption bricks develop high bond strength with both cement and cement lime mortars, when set either dry or wet.

4. High absorption bricks develop only low bond strength with cement and cement lime mortars, when set dry and the bond strength is marginally increased when these bricks are set wet.

Palmer and Parsons[73] studied how properties of bricks and mortars related to the bond and came to a similar conclusion as Collin. They observed that where suction rate of the brick was very low the bond strength was also low for all mortars; this increased with initial suction rate to reach a maximum strength when the rate of absorption approached 1-kg/m$^2$.min. After this, the bond strength declined as the rate of absorption increased.

Haller[102] found that the strength of the bond diminished with rising initial rate of absorption of bricks and considered that high quality bricks should not have suction rates greater than 1.5 to 2.0 kg/m$^2$.min.

The effect of water retentivity of the mortar upon bond was examined by Palmer and Parsons[73] in a series of permeability tests on small brickwork panels. They determined that walettes constructed of porous bricks laid dry were more watertight with mortars of high water retentivity than with mortars of low water retentivity. These results however do not agree with the conclusions drawn by Hogberg who found that a mortar with poor water retentivity gave a better adhesion to very absorbent bricks than a mortar with good retentivity. In a further experiment Hogberg[64] examined the effect of the ratio between binder and sand and found that with absorbent bricks the bond strength was usually improved when the amount of sand in the mortar was increased. This he argued was due to the fact that the binder-sand ratio and the water-cement ratio are closely interrelated. An increase in sand content requires an increase in water-binder ratio to maintain workability.

Hogberg concludes in his investigations that there are mortars that can be used regardless of suction background of the base material. He reported that binder-sand ratio of 1:5 and 1:6 gave best results.

Goodwin and West in their review of literature relating to bond show that there is abundant evidence in the literature to conclude that the rate of absorption of the masonry unit is the single most important factor affecting the bond.

The initial rate of absorption test appears to be generally accepted amongst workers as a reasonable and reliable test for clay bricks, however there is some uncertainty about its use for calcium silicate bricks and concrete blocks.

In consequence the failure of the IRS test to detect suction parameters which affect bond across an array of unit types, suggests that the test does not detect fully the water transfer across the bonding interface, which influences the bond formation mechanism.

It is considered in Chapter 7 that the water uptake properties of masonry units are inherently variable due to the pore structure of different types of units and indeed between units of the same constitution and kiln batch.

## 6.2 Experimental Programme

### 6.2.1 Experimental Design

A total of 400 fletton bricks from the same kiln batch were selected at random and their initial rates of absorption measured, as described in Section 6.2.2. Each population consisted of two individual sets of 200 units. The first set of samples were assigned to assess the effect of suction adjustment of units upon bond strength. The second set were used in a parallel experiment to determine the effect of non-suction adjusting the units. The resulting couplet bond strengths for both suction and non-suction adjusted units were compared.

In order to ensure an even distribution of suction rate throughout the sample population, each population was ranked in ascending order of IRS and then separated into quartiles of the range. This yielded eight distinct bands of IRS; four to be tested with no suction rate adjustment and four to be tested with suction rate adjustment.

The mean and standard deviation of each quartile were taken and bands selected which represented the mean ± 1 standard deviation of each quartile. This ensured that bricks from each band differed significantly from the adjacent band.

Each quartile range of the mean ± 1 standard deviation was compared with the corresponding quartile belonging to the alternative treatment, to determine that bands were not significantly different in their initial suction range. A further check was undertaken to affirm that each band was approximately normally distributed.

The experimental design, which is summarised in Table 6.1 below, was designed to be used with a two-factor analysis of variance (ANOVA).

Bricks were paired using values of initial rate of suction to produce 16-couplets per treatment combination, yielding 128 samples in total.

**Table 6.1: Experimental Design for Testing the Influence of Initial Rate of Suction on Tensile Bond Strength**

| | Initial Rate of Suction ($kg/m^2.min$) | | | |
|---|---|---|---|---|
| | 1st Quartile | 2nd Quartile | 3rd Quartile | 4th Quartile |
| **Non - Suction Rate Adjusted** | 1.13<br>0.95 | 1.36<br>1.21 | 1.61<br>1.47 | 2.60<br>1.80 |
| **Suction Rate Adjusted (prior to docking)** | 1.13<br>0.96 | 1.36<br>1.20 | 1.74<br>1.49 | 2.73<br>1.99 |

Values show ± 1 standard deviation from mean of each quartile range.

### 6.2.2 Initial Rate of Suction

The method for measuring the Initial Rate of Suction is described in Appendix H of BS 3921[66]. Each brick was physically marked with a specific number between 1-400, with an indelible felt-tip pen. The dry weight of each brick was then taken using a balance with resolution 0.1 grams. The balance was then reset to give the tare weight. The brick bedface was then immersed in water to a depth of 3-mm for a duration of 60-seconds. Upon removal, the bedface was dabbed with a damp cloth to remove excess surface water and then re-weighed. The tare weight, which indicated the weight of water absorbed by the brick in 60-seconds, was recorded. The equivalent mass of water removed by the brick was then returned to the measuring tank to ensure that the water level was maintained for the subsequent measurement.

The initial rate of suction was determined by dividing the mass of water absorbed per unit of bedface area. The bedface area was assumed as standard at 215mm x 102mm. The adoption of a standard bedface area, as opposed to measuring brick bedface dimensions individually, has previously been justified in Section 2.6.2. The resulting measurement yield the units of $kg/m^2.min$.

After measurement, bricks were dried back to their original dry weight for a period of 12-hours at 105°C in an industrial oven. This was undertaken to ensure that moisture from the IRS test was removed as rapidly as possible to discourage salt crystallisation in the pores, which could have influenced subsequent water uptake characteristics of the brick.

### 6.2.3 Suction Rate Adjustment

The only available guidelines on the procedure for suction rate adjustment of masonry units is given in Section 32.3 of BS 5628: Part 3[67], which recommends that bricks should be docked for not longer than 2-minutes just before use. In a realistic site situation, it is considered unlikely that units will be docked immediately before laying and will be permitted to drain prior to laying.

The objective therefore was to simulate as close as possible the site practice, while maintaining consistency. Units were submerged with their bedface uppermost in a tank of water with an approximate head of covering water of 300mm. The duration of submersion was 2-minutes, followed by a 10-minute drain period, with the bedface uppermost, to prevent continued saturation of the bedface by infiltration.

Those bricks for which the IRS fell outside ± 1 standard deviation of each quartile mean were used in a separate experiment to determine the effect of suction rate adjustment upon the measurement of the Initial Rate of Suction. Bricks which were suction rate adjusted prior to couplet manufacture were not tested to determine previously the effect of suction rate adjustment on IRS. Consideration was given that the degree of saturation, by docking units twice, could have profound influence upon the water uptake characteristics, for reasons mentioned 6.2.2 above.

### 6.2.4 Determination of Water Absorption

A sample of 10-units were tested for their water absorption using 5-hour boil test as described in Appendix E of BS 3921[66]. Equivalent populations were tested by 24-hour cold immersion and absorption under vacuum. The mean and standard deviations for each test are presented below in Table 6.2.

**Table 6.2: Unit Water Absorption**

| | Water Absorption [%] | | |
|---|---|---|---|
| | **24-hour immersion** | **5-hour boil** | **Vacuum Saturation** |
| **Mean** | 20.77 | 23.34 | 22.37 |
| **Standard Deviation** | 0.76 | 0.6 | 1.07 |

Results represent sample population of 10-samples for each treatment.

### 6.2.5 Couplet Manufacture and Testing

Couplets were manufactured and cured strictly in accordance with the procedure outlined in Section 2.2.2 and tested at 28-days age, in direct tension as described in Section 2.2.5.

Suction adjusted units were laid immediately after the 10-minute drain period. It was observed that there was no excess surface water on the bedface after the drain period and therefore units were not wiped with a damp cloth.

## 6.3 Experimental Results and Observations

### 6.3.1 Suction Rate Adjustment

Figure 6.1 shows the effect of 2-minute suction adjustment of the units (10-minute drain), upon the Initial Rate of Suction. In general the graph demonstrates that the effect of suction adjustment was to reduce the measured initial rate of suction by approximately 30%.

Both sets of data follow an S-curve relationship and the trends remain approximately parallel over the central range. It was observed that the bedface of bricks towards the higher end of the IRS range were heavily crazed, with large fissures. This may account for the high variability between non-suction adjusted and adjusted units at the higher levels of initial rate of suction.

### 6.3.2 Bond Strength

Figure 6.2 shows the relationship between Initial Rate of Suction and tensile bond strength at 28-days, for both non-suction and suction rate adjusted units. The X-value range bars show ±1 standard deviation either side of the mean suction rate for each quartile range. The Y-value range bars show ±1 standard deviation either side of the mean tensile bond strength for each treatment type, at each of the four quartiles of IRS.

During bond testing, it was observed that those couplets from the non-suction adjusted units depicted a higher proportion of top-plane failures or combined failures within the mortar than the suction rate adjusted units. Only one couplet from the suction adjusted treatment depicted a top-plane failure (2%), as opposed to nine for the non-suction adjusted treatment (14%).

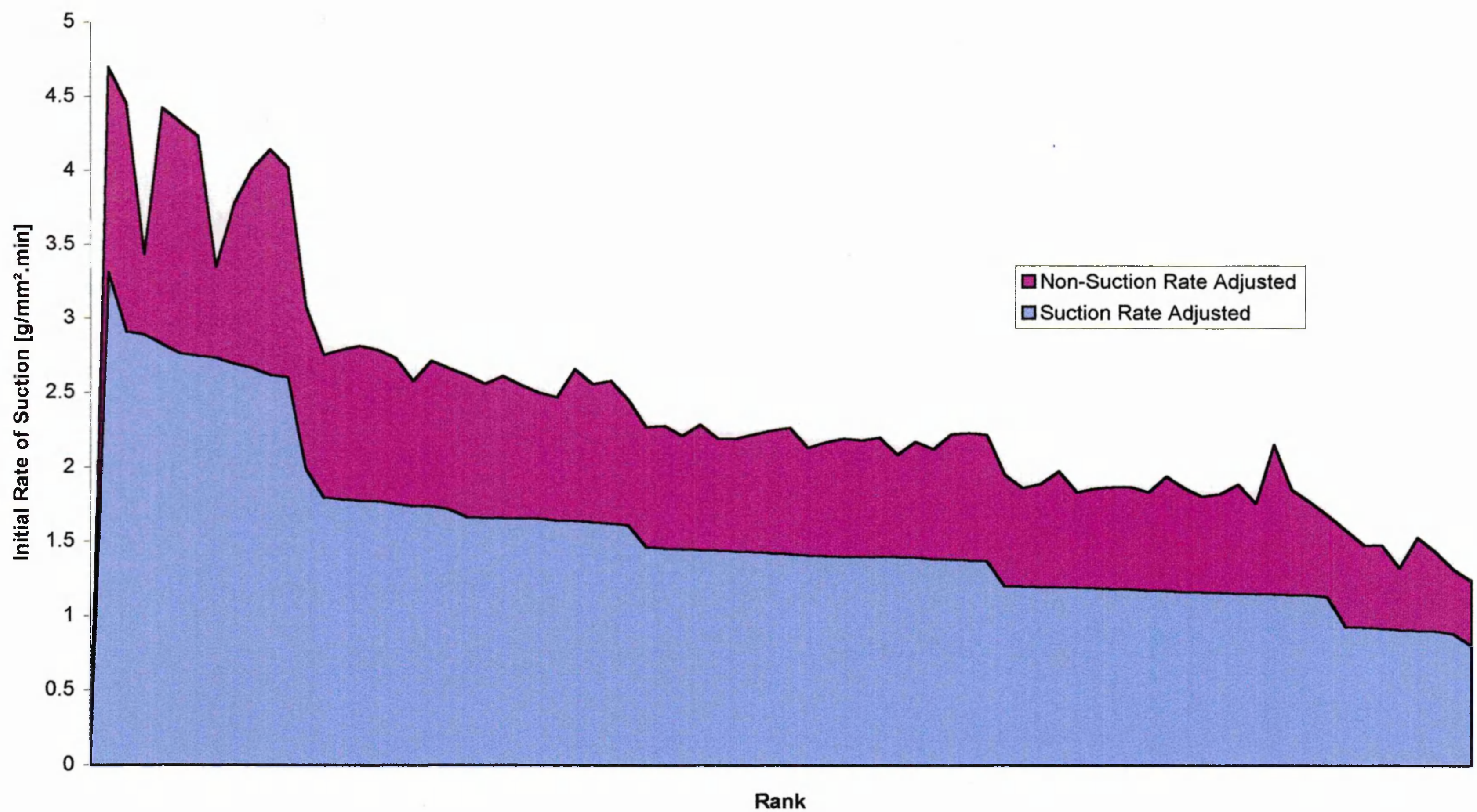



**Fig 6.1: Effect of Suction Rate Adjustment Upon Initial Rate of Suction**

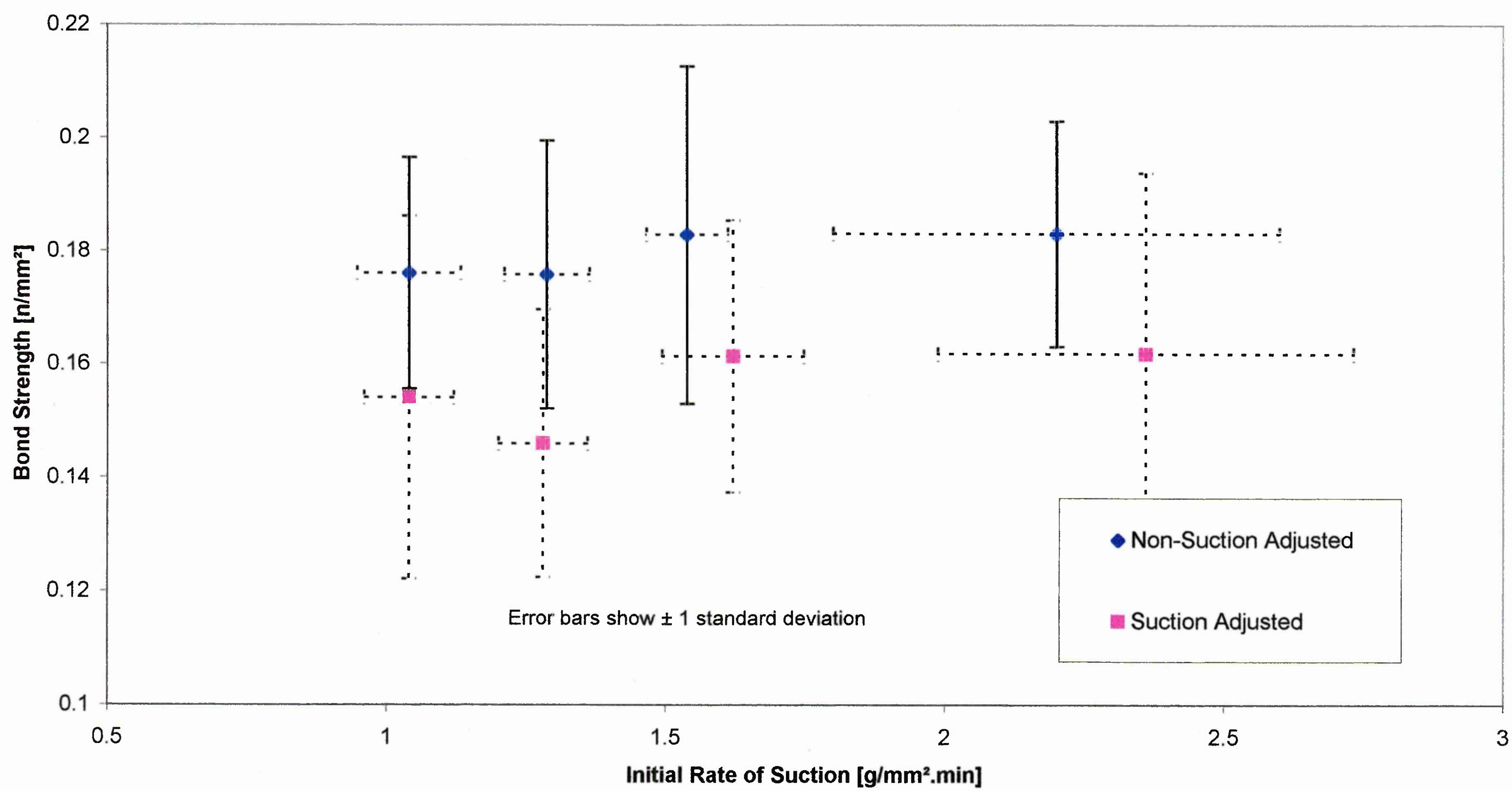



**Fig 6.2: Relationship Between Initial Rate of Suction and Tensile Bond Strength for Suction adjusted and Non-suction Adjusted Units**

## 6.4 Analysis of Results

### 6.4.1 Suction Rate Adjustment

In order to determine the effects of suction rate adjustment upon the measurement of the Initial Rate of Suction, a paired-sample design was used. The advantage of using such a design is that any variation between treatments may be attributed to the effect of that treatment rather than the background variability within the sample population. Since the direction of effect of suction rate adjustment was specific, it was possible to use a directional hypothesis, which halves the probability of a treatment having an influence by chance alone. A paired sample, one-tailed T-test was run using Minitab statistical analysis package.

<u>2-minute suction adjustment of the units results in a significant reduction in the measurement of Initial Rate of Suction (one-tailed test: $t=16.95$, d.f.$=75$, $p<0.01$)</u>

### 6.4.2 Bond Strength

To determine the effect of increasing rates of initial suction and the influence of suction adjustment, a two-way analysis of variance (ANOVA) was carried out using Minitab.

The advantage of this form of factorial experimental design is that it allows for the comparison of the main effect of several levels of IRS and the influence of suction adjustment, in combination. In addition, the presence of interaction between a specific level of IRS and the effect of suction adjustment can be investigated.

The results are summarised in Table 6.3 below.

**Table 6.3: Average Tensile Bond Strength Results for Varying Levels of Suction Rate and Suction Rate Adjustment**

| | Tensile Bond Strength ($N/mm^2$) | | | | |
|---|---|---|---|---|---|
| | Quartile Range of Initial Suction Rates | | | | |
| | 1st | 2nd | 3rd | 4th | **Average** |
| **Non - Suction Rate Adjusted** | 0.18 | 0.18 | 0.18 | 0.18 | **0.18** |
| **Suction Rate Adjusted** | 0.15 | 0.15 | 0.16 | 0.16 | **0.16** |
| **Average** | **0.17** | **0.16** | **0.17** | **0.17** | |

Averages shown are for both suction adjustment treatment and for each quartile level of IRS

The average bond strength for non-suction adjusted units ($0.180N/mm^2$) was significantly reduced by suction rate adjustment ($0.156N/mm^2$) (ANOVA: $F=26.24$, $df=3,120$, $p<0.001$). However, there was no significant effect of IRS level upon bond strength; neither was there significant effect of interaction between levels of IRS and suction rate adjustment.

## 6.5 Discussion of Results

### 6.5.1 Suction Rate Adjustment

The results shown in Figure 6.1 clearly demonstrate that suction adjustment of units has an effect upon the subsequent measurement of Initial Rate of Suction and that this was found to be highly significant by subsequent statistical tests.

Another apparent outcome of suction rate adjustment is that the variability in measured suction rates across the sample is reduced within the sample population; this is depicted by the smooth and gradual sloping line of the lower curve in Figure 6.1.

Over the central portion of the graph, the two lines run approximately parallel, demonstrating that there is a consistent reduction in measured values of IRS for this specific regime of suction adjustment. The extent to which varying levels of saturation or drain periods would have on the measurement of IRS have not been further investigated.

It was noted that an interesting phenomena occurred when units at the higher end of the IRS level were suction adjusted. As reported in Section 6.3, these units exhibited highly crazed bedfaces. Consequently, these samples demonstrated high initial rates of suction, which although reduced by suction adjustment, remained high and departed from the general trend of the curves. One explanation for this is that the large pores in the fissured bricks absorb large quantities of water during suction adjustment, but release this water more readily during the drainage period. Consequently, after a 10-minute drain period, the effect of suction adjustment upon measured IRS is not as discernible. A similar comparison could be drawn between clay units and aac blocks, perhaps explaining why these types of unit exhibit different water absorption characteristics.

This experiment demonstrates that the rate of water uptake, as measured by the Initial Rate of Suction test, is reduced if the units are previously suction rate adjusted by docking.

### 6.5.2 Bond Strength

The second experiment investigated the effect of IRS and subsequent adjustment of IRS upon bond strength. The results demonstrated significant differences in the levels of bond strength between bricks which were laid dry and those which were first suction adjusted. Surprisingly, the treatment of suction adjustment showed a marked reduction in bond strength. These results conflict with much of the current thinking embodied in the literature review in Section 6.2. The general consensus is that there is an optimum rate of absorption around 1kg/m².min[73]. Haller[102] concluded that bricks should not exceed 1.5 to 2.0 kg/m².min. A general approximation of the range of suction in this experiment lies between 1.5 to 3.0 kg/m².min prior to suction adjustment and between 1.0 and 2.0 kg/m².min after suction adjustment. Hence, the effect of suction adjustment of the units in this experiment was to bring the levels of IRS in line with recommended rates; however a corresponding reduction in bond strength was observed.

A further consideration is that there was no significant difference in bond strength detected between any of the quartile ranges for any particular suction adjustment treatment. A comparison between the fourth quartile range and the third, show that average levels of IRS were 1.0 and 2.25 kg/m².min, respectively. This means that the fourth quartile values would lie in a band comparable with units that had not been suction adjusted, while the first quartile range values would fall into the range of suction adjusted units. It is unclear why suction adjusted units should demonstrate a distinct reduction in bond strength, while units taken from extremes of the ranges demonstrated no significant difference in mean bond strengths. Perhaps even more surprising is that there was no measured effect of interaction between levels of IRS and suction adjustment treatment.

There are several explanations for the disparity in these results. Firstly, the departure from current understanding of the relationship between bond strength and initial rates of absorption could be explained by the type of tensile test adopted. As stated in Chapter 2, methods of flexural testing rely heavily upon compressive behaviour of the mortar bed. Consequently, parameters such as water-cement ratio which are directly influenced by the degree of water absorption of the unit, could influence results when testing in flexure differently to those attained from direct tensile bond testing.

A further consideration, which has been highlighted from the above experimentation, is that the Initial Rate of Suction test, while being widely accepted, is not sufficiently searching when attempting to identify the water uptake parameters necessary to explain the mechanism of bond development. It is apparent that the mechanism of suction adjustment of units has more bearing upon bond formation than extremes of suction backgrounds, as measured by the IRS test.

The experimental work was unable to establish a relationship between unit initial suction rate and bond strength.

The observed phenomena that non-suction adjusted units depicted a higher percentage of top plane failures may also indicate that there are mechanisms at work which are not fully detected by the simplistic IRS test.

## 6.6 Concluding Remarks

The experimental work discussed in Chapter 6, conflicts with much of the relevant theory and generally adopted site practice. Measurement of water transfer properties by the Initial Rate of Suction test have been shown to hold no relationship to magnitude of direct tensile bond strength, for the particular fletton brick adopted in this work. The practice of suction rate adjustment of units, while reducing measured IRS values to within reported desirable levels, was shown to have a detrimental influence upon the direct tensile bond strength. The marked presence of the significant effect of suction adjustment upon measured bond strength, associated with the absence of interaction for differing levels of initial suction rate, demonstrates that the IRS test did not distinguish water absorption parameters which contribute to bond formation.

Consequently, there may be further parameters of brick absorption characteristics, which may be inadvertently reflected by the IRS test, but which remain obscure. Researchers may have discovered optimum levels of IRS for the maximisation of bond, which have more to do with characteristics of a particular type of unit, rather than the measurement of IRS itself.

Work reported in the literature review demonstrates that while initial rate of suction tests may prove a suitable measure for the water uptake parameters of clay bricks, the test is not suitable for assessing the behaviour of calcium silicate or concrete blocks. Since each type of unit is expected to form a similar function in bonding with the mortar, then a test is required to measure the uptake characteristics of the unit, regardless of type.

Suction adjustment of the units should in effect assuage the degree of water absorbed subsequently by the unit during laying. However, the process of pre-wetting may have undetermined influence upon the capillary nature of the bedface pores. For example, pre-wetting may influence surface tension within the capillaries, leading to more rapid and instantaneous uptake of water.

The practice of suction adjustment, while having been exposed in this study as a detriment to bond development, does undoubtedly encourage good and more efficient workmanship, which in itself is advantageous to the development and future performance of the bond.

Once again, observations of a preferential failure plane, reported in the literature to be primarily top plane failure and shown in this work to be categorically bottom plane failure, suggests that there are mechanisms taking place which are not made apparent by the initial suction rate tests.

Despite the title of the Initial Rate of Suction, the test does not classify either the force of suction, the rate of flow or the quantity of water removed over the bond formation period. The work outlined in the following chapter develops further an understanding of the way water is absorbed by a particular unit and provides a means of monitoring the profile of the water absorption curve.

**References**

42. Kampf, L. Factors Affecting the Bond of Mortar to Brick. ASTM. Special Technical Publication No 320 p67; Symposium on Masonry Testing, 1963.

45. Sise, A., Shrive N.G., Jessop E.L. Flexural Bond Strength of Masonry Stack Prisms. Proceedings of the British Masonry Society April 1988 No 2 pp 103-107.

64. Hogberg, E. Mortar Bond. National Swedish Inst. for Building Research 1967.

66. British Standards Institution. Specification for Clay Bricks. BS3921. 1985

67. British Standards Institution. Code of Practice for Use of Masonry. Part 3, Materials and Components, Design and Workmanship. BS5628: Part 3: 1985

70. De Vitis N., Page A.W., Lawrence S.J. Influence of Age on Masonry Bond Strength: A Preliminary Study. Masonry International, Vol. 12, No.2, 1998.

73. Palmer, L.A., Parsons, D.A. A Study of the Properties of Mortars and Bricks and Their Relation to Bond. National Bureau of Standards Journal, Rp 683 1934.

75. Voss, W.C. Permeability of Brick Masonry Walls: a Hypothesis. Proceedings ASTM 33, 670, 1933.

78. Goodwin, J.F., West, H.W.H. A Review of Literature on Brick Mortar Bond. British Ceramic Research Association adapted from technical Note 308.

79. Davidson, J.L. Loss of Moisture From Fresh Mortars to Bricks. Materials Research Standards, part 1, p385. 1961.

81. Baker, I.O. A Treatise on Masonry Construction. John Wiley and sons, New York, 1909.

82. Barnstone, H. The Galveston That Was. The Macmillan Co., New York, 1909.

83. Moxon, J. Mechanik Exercises: or, The Doctrine of Handy-Works Applied to the Arts of Smithing, Joinery, Carpentry, Turning, Bricklaying. D. Midwinter and T. Leigh, London, England, 1703.

84. Shaw, E. Operative Masonry. Marsh, Capen and Lyon. Boston, Mass., p136. 1832.

85. Anderegg, F.O. Watertight Brick Masonry. Arch. Record 69, 201, 1931.

86. Gummerson, R.J. Hall, C. Hoff, W.D. Capillary Water Transport in Masonry Structures; Building Construction Applications of Darcy's Law. Chartered Institute of Building, Vol.1, No.1, Paper 2 p17, 1980.

87. Morgan, J.W. Brick Absorption. Architectural Science Review. p66-68, 1977.

88. Kirkham, D., Powers, W.L. Advanced Soil Physics. Wiley-Interscience, New York, 1972.

89. Murray, I.H. The Effect of the Absorption Characteristics of the Brickwork Background on the Adhesion of Renderings. BRE, Scot Lab.

90. West, H.W.H. A Note on the Suction Rate Test for Bricks. Proceedings of British Ceramic Society. No. 24, pp.21-31. Sept. 1975.

91. Anderegg, F.O. The Effect of Brick Absorption Characteristics Upon Mortar Properties. Proceedings of the ASTM, vol 42, p821. 1942.

92. Connor, C.C. Factors in the Resistance of Brick Masonry Walls to Water Penetration. ASTM Proceedings 48, 1020-1054, 1948.

93. West, H.W.H., Williams, A.N., Peake, F. The Effect of Suction Rate of Bricks on the Permeability of Mortar Beds. Ziegelindustrie International, 10, 421-425, 1976.

94. Collin, L.P. Laboratory Tests on Structural Assemblies of Brick and Tile. Canada Dept. of Mines Paper 766, Ottawa, 1935.

95. Palmer, L.A., Hall, J.V. Durability and Strength of Bond Between Mortar and Brick. Nat. Bur. Stands. J. Research RP290, 1931.

96. Fishburn, C.C. Water Permeability of Walls Built of Masonry Units. Nat. Bureau Stands. Rep. BMS 82, 1942.

97. Forkner, H.R., Hagerman, R.S., Dear, P.S., Whittemore, J.W. Mortar Bond Characteristics of Various Bricks. Bull. Virg. Polytechnic Inst. Eng. Exp. Stat. Series (7). 1948.

98. Houston, J.T., Grim, C.T. Field Quality Control Test for Wetting Brick in Masonry Construction. Journal of the Construction Division, ASCE, Vol.99,No. CO1, Proc. Paper 9839 pp. 5-10, July 1973.

99. Spradlin, W.H. The Building Estimators Reference Book.. pp. 443, 446 & 450 Frank R. Walker Co., Chicago, Ill., 1982.

100. Grimm, C.T. and Fowler, P.E. Effect of Brick Suction Rate on Bricklayer Productivity. Proc. North American Masonry Conf.., Arlington Texas, p8-1 to 8-18, June 1985.

101. Haller, P. Load Capacity of Brick Masonry. IBID, 129-149, 1969.

102. Haller, P. Die technsichen Eigenschaften von Backsteinmauerwerk fur Hochhauser. Verband Schweiz Ziegel-und Steinfabrickanten. Zurich 1959.

# CLASSIFICATION OF UNIT SUCTION PROFILES

## Chapter Summary

*Chapter 7 presents the hypothesis that it is the force of water abstraction by brick bed face capillarity which determines the true extent of water removal from the fresh mortar bed.*

*A unique method developed to monitor the continuous water uptake characteristics of the brick bed-face is presented.*

*The experimental results generate a continuous water absorption profile which can be described by an exponential decay curve; each curve being characterised by a time constant.*

*The method provides a means of identifying the rate of change of flow and resulting force function, with which water is potentially extracted from the retentive mortar background.*

*Various types of clay units and treatments are characterised using the time constant and the resulting water absorption characteristics are considered in relation to bond strength performance.*

# 7.0 Introduction

The survey of unit suction characteristics, discussed in Chapter 6, identifies that the Initial Suction Rate test, outlined in BS3921[66], is not sufficiently searching of those water absorption parameters which contribute to the formation of the brick-mortar interface bond.

The results demonstrate that the treatment of suction adjustment of units by "docking" has a significant effect upon the measured bond strength. While the treatment of suction adjustment reduces the measured water absorption, there remains no correlation between bond strength and water absorption of units when measured by the Initial Suction Rate test. The absence of interaction in the two-way factorial experiment in Chapter 6, highlights the fact that the process of suction adjustment must influence the mechanism by which water moves into the unit. This poses the question whether there are processes at work which influence the way water is absorbed by a unit which are not made readily identifiable by traditional methods of measuring water absorption.

Quantities of water abstraction by unit suction from a free water surface, whilst providing a measure of a units water demand, do not fully describe a units potential to extract water from a retentive wet mortar bed. It is considered that it is the rate of water absorption, rather than the quantity, which is indicative of the suction force necessary to extract the excess mix water from the mortar bed.

This study postulates that the critical parameter controlling bond strength is the removal of the excess mix water within the first few seconds of bond formation. The instantaneous suction force, which occurs upon immediate contact between the brick and the wet mortar, contributes to the critical transfer of water across the brick-mortar interface. If suction forces are high, depicted by a steep slope on the water absorption curve, then rapid plastic shrinkage of the mortar will ensue, with resulting reduction in the potential long-term drying shrinkage. If the force of suction is diminished, then less of the excess mix water will be removed from the mortar which will lead to protracted levels of drying shrinkage at the brick-mortar interface.

Consideration of the literature relating to unit suction in Section 6.1, reflects an optimum level of Initial Rate of Suction for the maximisation of bond strength to lie generally between 1.0 and 2.0 $kg/mm^2.min$; this value equates to a level of water uptake from a free water surface of between 22-grams and 44-grams, for a standard format brick bed-face area.

For the array of mortar mixes studied in Chapter 3, it can be shown that the mix water contributes approximately one-quarter of the volume yield of the wet mortar. Therefore, in a newly laid couplet mortar joint of 10-mm uniform thickness, the amount of water present in the mortar in the fresh state is between 38-grams to 43-grams. If it is accepted for present that excess mix water is distributed equally by brick suction forces to both the lower and upper unit, then the potential water available at the bonding interface is in the region of 20-grams. Consideration was given in Chapter 3 that only one-third of this mix water is necessary for cement hydration, leaving the remainder available to assuage the demands of brick suction forces. This suggests that the desirable level of water to be removed by brick background suction to be approximately 12-grams, which corresponds to an Initial Rate of Suction of 0.5 $kg/mm^2.min$.

Clearly there exists considerable imbalance between the potential water which can be absorbed by a unit from a free water surface and the water available within the mortar bed. Evidently, it is the force of suction which reflects a units ability to remove water away from the retentive background of the mortar bed. Hence the Initial Suction Rate test does not reflect true in-situ suction performance.

The unit suction characteristics, as determined by the Initial Suction Rate test, are defined by two points on a graph; namely the origin and the mass of water absorbed after an arbitrary 60-seconds. Due to the influences of bed-face surface wetting and variation of pore structure throughout the unit, it is improbable that these two reference points on the absorption graph will be joined by a straight line. Of relevance to this study therefore is the shape of the water absorption profile which connects these two points, since it provides an indication of the distribution of flow with time and therefore the velocity and suction force.

Applying a derivation of Darcy's Law for flow in an unsaturated porous media, it can be shown that the level of water uptake is approximately linear with the square-root of time. This has been confirmed by many researchers working in the area of water absorption characteristics of masonry and their findings are discussed in detail in the review of literature in section 7.1.

Traditional methods of measuring water absorption with time require the removal of samples from the water surface at specific time intervals. Consequently interruption of the suction process causes disruption to capillary rise and there is no guarantee that the process of water absorption remains consistently linear with the square-root of time under continuous capillary rise. A further complication induced by intermittent measurement is that repeated surface wetting and surface layer pore filling of the submerged bed-face may contribute significantly to the measured water uptake.

The experimental method developed for this work and described in Section 7.3, provides a unique means of measuring the continuous and uninterrupted water absorption of a brick bed-face. The resulting experimental data generates an exponential decay curve, the shape of which is typical to many curves depicting natural and physical phenomena. One such example is the shape of the curve produced by the measurement of the charge held on a capacitor, when charged through a resistor in a simple resistance-capacitance AC electrical circuit.

The likening of water movement through porous media to an electrical circuit is perhaps justified when one considers that moisture measurement techniques used in industry apply similar methodology to determine moisture contents of materials such as cotton and paper. Harbert[103] explains that in its simplest form, water absorbed by porous media serves a similar function to dielectric between two plates of a capacitor and argues that resistance and capacitance metres provide the basis for most moisture meters in industry.

The equation used for describing the charging of a capacitor can be adapted to describe a units continuous water absorption profile curve and takes the form shown in Equation 7.1 below. The equation permits each water absorption curve to be characterised by two key parameters, namely a maximum value of water absorbed ($M_{max}$) and a time constant (RC).

$$M = M_{max}\,(1 - e^{(-t/RC)}) \qquad \text{Equation 7.1}$$

The total water absorption ($M_{max}$) into the surface layers of the brick bed-face is reached at the point when the absorption curve becomes asymptotic; this is distinguished from the condition of partial saturation, which involves water transfer through the body of the unit over prolonged time period.

Referring to Equation 7.1, when time (t) equals RC, the mass of water absorbed corresponds to two-thirds of the maximum water absorption ($M_{max}$). Hence the time constant (RC) can be determined from the experimental results.

The determination of the time constant is of particular importance to this study, since it provides a means of characterising the "shape" of the water absorption profile, or more specifically, the distribution of mass flow rate and correspondingly the suction force. A correlation can then be made between the characteristic profile of the water absorption curve and the bond strength performance.

## 7.1 Discussion of Literature Relating to Water Absorption of Porous Materials

There exists a wealth of literature concerned with water transfer through porous building materials. In the main, the literature has focused specifically on water absorption characteristics which influence the materials durability behaviour when exposed to the outer climate. The phenomena of water transfer is of particular importance in masonry and concerns durability parameters such as frost, damp, interstitial condensation, salt staining and rain penetration.

The investigation of water absorption parameters and their relation to bond strength considers the mechanism by which water is removed from the wet mortar bed into the surface layers of the brick bed-face by brick suction forces. Studies that focus on durability parameters are concerned essentially with how the water moves within the body of the material, influenced by the distribution of pore size and shape. Consequently, the two different studies depart similarity at this point; however studies of the overall mechanisms of water transfer help to create a global understanding of the processes under investigation in this chapter.

Much of the work concerned with the study of flow in saturated and partially saturated porous materials, develops from the theory of soil mechanics.

Gummerson, Hall and Hoff[86] in their study of capillary water transport in masonry structures, emphasise that the water absorption porosity measures only the water holding capacity of a porous material and does not define the way in which water moves into such a material. They suggest that while a complete understanding of water movement in different materials requires knowledge of their microstructure, it is not necessary to adopt a microscopic approach in order to characterise their water transport properties.

Applying the theory of unsaturated flow to the movement of water in building structures, the authors recognise that several separate forces act on water held in a porous material. These can be defined as follows:-

> Gravitational forces, characterised by a gravitational potential normally expressed as the height of the pore above some defined datum.
>
> Suction forces, characterised by a capillary potential defined as the energy to remove a unit mass of water from a partially saturated porous solid to a free state at the same level.
>
> Capillary potential, which can be described as the hydraulic tension head which can be sustained by a capillary of diameter equal to the mean diameter of the pores at the air water interface.

The authors propose that Darcy's fundamental law, which describes the flow of water through saturated porous material (saturated flow), can be extended to the study of partially saturated materials (unsaturated flow), and provides a rationale for the study of a variety of water movement processes occurring within the building fabric. In unsaturated flow, capillary potentials operate in addition to possible external forces and hydraulic pressures.

Such an example of the application of Darcy's Law is given by Kirkham and Powers [104], who provide an analysis which leads directly to a simple square-root of time relationship, both for the advance of the wetting front and the quantity of the water absorbed.

The total distance x, advanced by the wetting front at time t:- $x = Bt^{½}$

B is a constant, termed the Water Penetration Coefficient

The total amount of water I, absorbed at time t:- $I = St^{½}$

S is a constant, termed Sorptivity or Water Absorption Coefficient

Darcy's law, essentially defines the hydraulic conductivity of the material. If gravitational effects are ignored, then a square-root of time dependent relationship of the capillary absorption of water by masonry materials can be shown. Gummerson, Hall and Hoff state that gravitational effects are not discernible in short-term capillary rise experiments.

The authors conducted an experiment to measure the cumulative inflow of water into two types of building material, a clay brick and an autoclaved aerated concrete block (aac). Both materials demonstrated a square-root of time dependence, however the aac block showed half the sorptivity of the brick. These results are in broad agreement to the results found by Bomberg[105]. The reason given for the aac blocks low sorptivity, despite its high porosity, is that the large aeration pores exert a very low suction and the water drawn into the solid by capillary action is essentially held in the fine pored matrix.

Murray[89] studied the effect of the absorption characteristics of the brickwork background on the adhesion of renderings. He took the Sorptivity factor to be the gradient of the straight line, obtained when I, the volume of water absorbed per mm $^2$, is plotted against the square root of time (minutes). Murray found that higher adhesion values were obtained for render on bricks with greater sorptivity. It must be recognised that these results were for the stretcher face of clay commons, which may exhibit very different absorption characteristics to the brick bed-face. Murray also determined that when sorptivity is high, the adhesion of render is better, so for dry bricks the adhesion is good. However, suction adjusted or "docked" bricks have low sorptivity, typically below 0.5 mm/min$^{½}$, and therefore adhesion of render is poor. Murray's results can be compared to the experimental results in Chapter 6, which showed that suction adjusted bricks produced lower bond strengths.

Murray quotes typical values of sorptivity for Scottish clay commons ranging within 0.6 to 0.95 mm/min$^{½}$ which are in broad agreement to the results quoted by Gummerson et al[86].

Morgan[87] in his study of brick absorption, investigated what he termed to be the "peculiar" bonding characteristics of some types of unit, particularly concrete and calcium silicate units. Morgan measured the amount of water uptake through unit bed-faces, submerged to a depth of 3-mm, for a number of immersion times. He found that the relationship between water absorption and the corresponding time interval was linear with the cube-root of time. Morgan argues that his choice of a cubed-root relationship was influenced by the S"-shaped trend of the data points and the fact that curves should, in theory, pass through the origin. The cubed-root time dependence is contrary to the relevant theory of Darcy's Law as discussed above, however West[90] also reports a cubed-root relationship in his study of Palmer and Parsons.

Morgan determined the instantaneous absorption rates from the first derivative (with respect to time) of the fitted lines. Although Morgan does not conduct bond strength tests in his study, he maintains that the instantaneous absorption rates are most relevant to the problem of brick-mortar bond strength. He considers that both the early absorption in the first few minutes and long term absorption rates are significant. He points out that instantaneous absorption rates at 1-minute are not equivalent to initial rates of suction test results, but are more likely to be typical of on-site brick absorption characteristics. The absorption rate was assumed to reach zero at the end of the straight line time period. However, although Morgan terms early absorption rates as instantaneous, the first measurement was taken at 60-seconds which does not reflect the true instantaneous water transfer into the material. Morgan remarks that the lines of best fit do not pass through the origin due, he ascribes to the experimental error as the rapid absorption rates at early times make experimental technique very critical in this region.

Morgan examined the early water absorption behaviour of calcium silicate and concrete bricks. He observed that these types of unit have the lowest early absorption rates, despite their medium to high values of initial suction rate. In addition, time periods for which significant suction rates for these types of units persist are much longer than corresponding time for clay units. Morgan considers that the high affinity for water exhibited by these types of bricks does not result solely from capillary mechanism's dominant in clay brick types, but may be caused by aggregate type in concrete, distribution of voids and even chemical composition. In view of the bond characteristics of these brick types, observed in previous laboratory tests, Morgan suggests that absorption characteristics of low initial absorption, coupled with relatively high long term absorption levels, is a significant source of problem associated with the development of satisfactory bond strength for concrete and calcium silicate units. He argues that sustained absorption rates, in the later stages will tend to dehydrate the mortar in the vicinity of the bond region. Morgan concluded that the Initial Rate of Suction test may not be such a reliable indicator of desirable mortar properties for calcium silicate and concrete units, as it is for clay bricks.

Morgan maintains that the absorption characteristics of concrete and calcium silicate bricks indicate the need to develop mortars with comparatively low water retention for formation of good bond, but with high water content capacity to prevent dehydration by prolonged water absorption.

Schwarz[106] studied capillary water absorption of building materials and confirms a square-root time dependence. Schwarz uses the term Water Absorption Coefficient (WAC) rather than sorptivity to describe the slope of the straight line plot of water absorption against the square-root of time. Schwarz characterises the WAC as the change with time of water absorption of a material from the dry state (in equilibrium with surrounding atmosphere), until it is wetted right through but acknowledges that it involves the limited condition of there always being excess water at the suction face. When the amount of water offered to the material is restricted, as in the case of absorption from a wet mortar bed, then the capillary water absorption is no longer determined by the WAC, but rather by the available quantity of water.

Kunzel and Schwarz[107] recognise that determination of water absorption by capillary forces work in opposition to the forces of gravity, but from previous tests were able to show that these additional forces have no measurable effect on suction in finely porous media.

In this study, the authors observe that a graph of water absorption as a function of the square-root of time yields two sections which differ distinctly in their slope. The first section represents water absorption through the sample, which is initially dry and the second part of the curve after a point of inflection describes the water absorption of the sample once wetted throughout. The authors refer to the moisture content as the water capacity (moisture content by volume), which they suggest is indicated by the point of inflexion on the curve. Bomberg[105] uses the term "bubbling point". Murray also reported that the initial sorptivity rate over the first 30-minutes was on average greater than the rate for longer periods.

Schwarz[106] noted that the water absorbed by capillary action, when plotted against the square-root of time, did not always pass through the origin. He observed that this was particularly evident for coarse pored materials and attributed it to starting up states in the first few moments of absorption.

Anderegg[91] investigated the effect of brick absorption characteristics upon mortar properties. Anderegg used bricks from different parts of the same kiln to set the water absorption criteria and measured water extraction from lime mortars. Anderegg refers to observations by Stull and Johnson[108] that considerable variations in pore structure are often encountered in different parts of the same brick.

Anderegg observed rapid rise of water in the dry-press bricks and attributed this to the presence of relatively large pores. The very slow absorption of the hard-burned bricks is due to a finer pore structure.

Anderegg noted that the type of brick having the highest initial rate of absorption removed only a very small amount of moisture from lime mortars. Amongst the more absorptive bricks, most moisture was removed by some of the bricks having lowest initial absorption. It was also observed that in order to achieve mortar set, more water had to be abstracted by the softest burned brick than by the hardest burned brick, suggesting the presence of a marked differences in moisture gradient within the mortars in contact with different bricks.

The higher the absorption rate, the greater was the apparent tendency to form a highly congealed layer at the surface of the mortar. The rest of the mortar could be readily peeled from this congealed layer. Anderegg maintains that moisture is removed so rapidly from the surface of the mortar by some bricks of high initial suction capacity, that a condensed layer is formed having a permeability varying inversely with the initial suction.

The formation of a very dense congealed layer next to the bricks resisted the transmission of moisture from the centre of the mortar joint. In-spite of this congealed layer, there was often sufficient suction force to continue the abstraction of moisture over a considerable period of time.

Jansson[109] plotted the amount of water absorbed against the square root of time and found it to be reasonably linear. However, he holds the view that the standard initial suction rate test provides little information about the properties of the brick. Sneck[110], like Jansson also queried the value of the initial rate of absorption from a free water surface as a criterion which could have influence on the properties of the bond. He also found that calcium silicate bricks with a high initial absorption rates, drew less water from the mortar than other calcium silicate bricks which had lower initial absorption rates.

Sneck[111] in a further study considers that the suction exerted depends on the water absorption of the masonry unit, the rate of absorption and the capillary suction force. The water absorption gives an indication of the amount of water that the brick is capable of removing from the mortar and the rate of absorption tells generally how rapidly the water is removed. The capillary suction force becomes important when the masonry unit consist of material with very fine pores, which is the case for many calcium silicate bricks. Sneck argues that these fine pores exert a strong suction for a long time, which is quite different from the properties of clay bricks.

West[90] examined a number of different clay bricks and two calcium silicate bricks and found that the later gave a curve of suction against time approximating to low water absorption clay bricks.

De Vitis, Page and Lawrence[70] suggested that gravity plays a role in the transport of water through the plastic mortar, with greater amounts of water being absorbed by the bottom unit within the couplet. However the author's predictions are not based on measuring water abstraction from mortar, in which case it is probable that suction forces would be greater in the upper brick, where capillary rise works in opposition to gravity, rather than in the lower brick where capillary suction operates under conditions of partial saturated flow due to initial infiltration of water leaving the wet mortar bed.

Hall[112] describes three possible test configurations to measure the rate of uni-directional water absorption in a porous medium:-

> Horizontal inflow, where absorption is affected by hydrostatic forces and there are no significant influences of gravity
>
> Infiltration, where absorption is partly due to capillary suction and partly to gravitational forces
>
> Capillary rise, where the effects of capillary and gravity are opposed.

Water transfer through porous materials has been comprehensively investigated in concrete research where it is associated with durability issues such as freeze-thaw damage, sulphate attack, alkali-aggregate expansion, chloride ingress, reinforcement corrosion and carbonation. Investigation of durability parameters tend to require determination of water movement through the body of a material over a sustained period of time, whereas water transfer from a fresh mortar bed across the bonding interface is associated with the initial effects of suction over a short time period. Notwithstanding, a review of literature relating to water transfer in concrete is beneficial to the understanding of the way in which water is drawn from a fresh mortar bed.

Khatib and Mangat[113] studied absorption characteristics of concrete as a function of location relative to casting position. The authors determined the weight of water absorbed per unit area by capillary rise and plotted this against the square-root of time. The initial slope of the straight line was taken as the water absorption coefficient, WAC. Large differences in water absorption coefficient were found between the top and bottom parts of the cubes. The authors observed that WAC for the top surface can be over three times that for the bottom surface.

Khatib and Mangat argue that the WAC test is much more sensitive to changes in porosity and pore structure than shallow immersion water absorption tests. This, they suggest is because the WAC test measures the rate at which water is drawn into a single face, by capillary action, whereas the latter measures only the total absorption. They cite an example whereby specimens may have similar porosity and may give the same value of water absorption, but may demonstrate different values of WAC, dependent upon the diameter of the capillaries. The authors conclude that whereas the water absorption test is a function of porosity of the concrete, the WAC ($g/mm^2s^{1/2}$) is a function of porosity and pore structure.

The Initial Surface Absorption Test (ISAT), for measuring the surface permeability of concrete, is outlined in BS1881 Part 5[114]. Although the test essentially gives a permeability measurement, as opposed to a measurement of water absorption by capillary rise, it is perhaps suited to adaptation in order to measure loss of moisture from wet mortar to the lower unit in the assemblage. The test has been the subject of rigorous evaluation by Levitt[115], who notes that the test procedure relatively lacks accuracy when rates of absorption are very high or very low. BS 1881 specifies the use of a graduated capillary tube and stop-watch to monitor the movement of the meniscus at specified time intervals.

Levitt used Poiseuille formula to show that if the absorption process is controlled by capillary flow into the substrate, the rate of absorption after elapsed time t, will be given by:-

$$\text{Rate} = Ct^{-m}$$

where C is a constant and m is theoretically equal to 0.5.

In practice (m) is found to lie between 0.3 and 0.7 for concrete. A high value of power in this experiment reflects the rapid decline in absorption rate and low overall absorption.

It is conceivable that the deviation from the square-root of time relationship, found by Levitt could be as a result of the measurement of water uptake acting in unison with gravity.

Sabir, Wild and O'Farrell[116] developed a uni-directional water sorptivity test for mortar and concrete. The authors argue that there are two mechanisms controlling the uptake and transport of water. Permeability, which is the measure of flow of water under pressure in a saturated porous media and sorptivity, which characterises the materials ability to absorb and transmit water through it by capillary suction. The authors argue that sorptivity is dependent upon initial water content and its uniformity throughout the test specimen. They add that sorptivity does not take place in saturated specimens and consequently docked units may not absorb water by capillary action. They suggest that some materials with extremely coarse pore structures experience little capillary suction and may show significant deviation from linearity.

The authors found in their investigations that the point of origin should not influence the determination of the slope of the graph, due to initial increase in mass of the specimen caused by preferential filling of the open surface pores on the inflow face and sides of specimen when submerged.

Parrott[117] investigated water absorption in cover concrete and observed also that the depth of water penetration usually increases linearly with the square root of wetting time in many porous materials, provided that porosity and initial moisture content are uniform throughout the test specimen and that water penetrates as a sharp liquid front.

Parrot determined that methods of curing may significantly affect the relative values of absorption rate of the hardened concrete. Hydration of cement can be restricted in cover concrete and the average capillary porosity can be higher than that of the interior concrete. Carbonation will normally cause a counterbalancing reduction of capillary porosity in outer layers of cover. Thus linear relationship between absorption depth and square-root of time may not be obtained under field conditions. Parrot found that the exponent for the power law function to be 0.32 rather than 0.5. He also observed that the shape of each absorption curve appeared to be geometrically similar.

Winslow, Kilgour and Crooks[118] in their study of the durability of bricks used mercury intrusion porisimetry to examine pore size distribution in different bricks. They determined that the extent of vitrification (indicated by the smooth glassy areas) is lowest in the Fletton. The predominant feature of the fletton is of fine grains, lightly bonded together, reminiscent of the semi-dry granular consistency of the clay used in their manufacture. Engineering bricks are characterised by a much higher degree of vitrification together with fewer, coarser pores than the Fletton. Such pores are probably also largely discontinuous which would account for the lower water absorption of these highly durable bricks. The changes in pore structure are related to the phase changes that occur as a result of firing. Decrease in porosity with increased firing temperature was accompanied by a significant increase in diameters of the modal pore sizes. As more of the body fuses, the larger pores seal over so that total open porosity is reduced. Extensive firing should reduce a brick's pore volume and transform more of that volume to larger pore sizes as the raw materials vitrify.

Whiteley[119] in his observations of the difficulty in painting Fletton bricks, noted that failure of paint film adhesion occurred over kiss marks, which are areas of the brick surface which have been in contact in the kiln and appear darker, smoother and often slightly glazed. Whiteley concluded that the coarser pore structure of the kiss marks allows greater movement of moisture and relates this to problems encountered with paint adhesion by increased tendency of efflorescence. Kiss marks are often located on the brick stretcher face and enhance the appearance of the brick. Kiss marks located on the bed-face have increased pore diameter and could dramatically influence water extraction from the fresh mortar bed.

Ritchie[120] examined laminations in bricks and although he did not make direct correlation with water absorption, predicted that capillary fissures caused by lamination, could contribute to water absorption. Ritchie argues that the flow of clay past the auger and through the die frequently produce laminar voids. Such voids may result from the inability of the clay to unite after it has been separated. Another cause is friction between the metal die and the clay; differential rate of flow through the die which sets up shear stresses, producing cleavage planes. Ritchie argues that the action of cutting wires is to smear the clay, thereby obscuring any laminations within the body of the material.

Increasing the maximum firing temperature invariably produces a greater degree of vitrification and therefore a reduction in fine pore content. May and Butterworth[121] note that in general, increasing firing temperature result in an increase in pore size and a reduction in total porosity. The authors suggest that sometimes very fine pores are either filled preferentially, or alternatively not filled at all.

Winslow, Kilgour and Crooks[118] recognise the fact that large pores, while contributing to the total volume, are unlikely to be full of water since they drain more readily.

The literature discussed above is pertinent only to the understanding of the transfer of mix water across the bonding interface. The research demonstrates that tests which measure the quantity of water absorption by a unit from a free water surface, do not necessarily identify water abstraction from a retentive mortar bed.

It has been reported that units with large pore size, such as concrete blocks or highly fired clay bricks, remove very little water from the mortar, however remain capable of extracting relatively large quantities of water from a free water surface. The reason given for this phenomena is that larger capillaries exert reduced suction force.

The literature also identifies that suction adjusted units exert less suction force and this is explained by the fact that capillary suction does not occur under conditions of saturated or partially saturated flow.

Several workers have plotted the water absorption against the square-root of time and have found that the relationship is linear. While workers have postulated that the gradient of the line or sorptivity is related to the pore structure, few workers have volunteered any explanation as to why sorptivity or flow rate should influence the bond development.

The work presented in the following sections examines the phenomena of sorptivity and investigates its relationship with the bond strength.

## 7.2 Characteristics of Sorptivity

Cumulative water absorption by a unit bed-face from a free water surface, with respect to time, has traditionally been measured using an adaptation of the Initial Suction Rate test, described in Section 6.2.2. The mass of water absorbed by the unit is determined by weighing the sample at specific time intervals. A plot of the mass of water absorbed against the square-root of each respective time interval generally yields a linear relationship. The gradient of this line describes the sorptivity, which is a function of flow rate [grams/second $^{½}$].

### 7.2.1 Determination of Sorptivity From a Free Water Surface

Sorptivity values usually describe the water uptake characteristics of a porous unit from the air dry condition to a point of partial saturation. Consequently, the time intervals between measurements tend to be relatively large and are taken over several hours. For this study, it was desirable to assess the water absorption characteristics within the first 60-seconds of contact between the bed-face and both a free water surface and a fresh mortar bed. Time intervals of 10, 20, 30, 40, 50 and 60 seconds were adopted.

Due to the requirement to measure water absorption over a short time period, with only 10-second interval between measurement, it was necessary to use different units for each point of measurement. In order to facilitate this a range of 100-units were tested for their Initial Rate of Suction and a total of 9-units, with similar corresponding values of IRS selected to represent mid-range suction of the population. Each unit was then cut in half, to provide additional representative bed-face areas; the IRS test was repeated to ensure that initial rate of suction characteristics did not differ across the bed-face. In total, 18-half units were tested, generating three readings for each of the six time intervals.

Each time interval was conducted as a separate measurement. Units were first tested for water absorption from a free water surface, with the bed-face submerged to a depth of 3-mm. After the corresponding time interval, the units were removed and the surplus surface water wiped with a damp cloth and the mass of water absorbed, determined by weighing. The samples were then oven dried at 105°C until a constant weight was attained in preparation for the following experiment.

### 7.2.2 Determination of Sorptivity From a Fresh Mortar Bed

The same samples were then tested to measure their water absorption from a fresh mortar bed. A 1:1:6 cement:lime:sand mix, as described in Section 3.2 was adopted to represent the wet mortar background. A bed of mortar was laid onto a non-absorbent glass plate between two parallel, 10-mm deep aluminium gauging bars, set 110-mm clear distance apart. The mortar bed was then struck flush with the top of the gauging bars using a palette knife. Three mortar beds were created in this way, from three separate mortar mixes. The six half units were then placed on each mortar bed in rotation, corresponding to each specific time interval. The units were placed on the mortar bed using self-weight pressure only. After each corresponding time interval, the samples were removed from the mortar bed and weighed to determine the mass of water absorbed. It was noted that it was not necessary to wipe the unit surface before weighing, as there was no visible excess surface water. Furthermore, it was not considered necessary to use a separating membrane, such as gauze, between the brick and the mortar, since there was no visible evidence of mortar particles adhering to the brick surface; consideration was given to the fact that the presence of such a separating membrane could contribute to water abstraction from the mortar.

### 7.2.3 Sorptivity Results

Figure 7.1 shows the relationship of water absorption of a typical half-Fletton unit with time for both a free water surface and a 1:1:6 cement:lime:sand wet mortar surface. It should be noted that the y-axis values have been multiplied by a factor of two to reflect water absorption for a full-brick bed-face area. It is considered that for the purposes of comparing results, since all bed-face areas are approximately equal, that values of water absorption reported in grams are easier to interpret at this stage.

### 7.2.4 Calculation of Sorptivity

Consideration of the unsaturated flow theory as first derived by Darcy, reported in Section 7.1, provides theoretical basis that the cumulative mass inflow of water obeys a linear relationship with the square-root of time. Figure 7.2 demonstrates such a relationship for both water absorption from a free water surface and a fresh mortar surface.

The slope of this line, which for the purposes of determining sorptivity, is assumed to pass through the origin, is a function of the mass inflow rate [grams/ second$^{½}$]. This characteristic is termed Sorptivity and is sometimes referred to as the Water Absorption Coefficient.

### 7.2.5 Discussion of Sorptivity Results

Figure 7.1 shows that fifty percent of the mass of water absorbed from the mortar over a 60-second period can be absorbed within the first 10-seconds upon contact with the mortar. The same trend is reflected by the profile of absorption from the free water surface, however water absorption from the water surface shows that significant water extraction continues after this initial stage. Continued water abstraction from the mortar is not demonstrated and equilibrium is reached within 60-seconds. The two different profiles reflect the difference between the retentive background of the wet mortar bed compared with the free water surface.

The results show that a significant proportion of the available free water is removed within the first few seconds upon contact with the mortar, suggesting that the instantaneous rate of water absorption is critical when considering a units potential to extract water away from the retentive background of the mortar. The same rapid extent of water removal is demonstrated by water absorption from the free water surface. Similarities in the behaviour of the absorption curves between mortar and water would suggest that it remains practical to measure and characterise instantaneous suction parameters by measuring a units response to a free water surface, provided that it is acknowledged that the retentive nature of the mortar has an influence on the gradient of the curve and the maximum amount of water that can physically be extracted.

Figure 7.2 shows that the sorptivity from the mortar is approximately one-half that from the water surface, with values of sorptivity 0.56 and 1.2 [$g/s^{½}$], respectively.

Sorptivity is considered to be constant throughout the body of the porous media. This assumption may be accurate when investigating water absorption characteristics associated with the durability of the material, concerned largely with water intake up to a point of partial saturation. However, water absorption from a fresh mortar involves limited quantities of water which will be distributed into a finite boundary layer of the bed-face, the characteristics of which may not necessarily reflect the body of the unit as a whole. For example Murray[89] argues that due to the nature of firing, capillaries and pore structure close to the bed-face are very different to the body of the brick and therefore influence brick absorption accordingly.

Calculations of sorptivity are dependent upon accepting two important principles:-

> The relationship between water uptake and the square-root of time remains linear despite the fact that traditional methods require disruption to capillary hydraulic suction forces for the purposes of periodic weighing.
>
> The initial start point lies on the origin.

In the absence of a method to continually monitor the water uptake, there is no guarantee that the process holds to a linear function due to the effects of capillary suction disruption.

Furthermore, submersion of the bed-face to a depth of 3-mm causes surface wetting not only to the bed-face area, but also to the sides of the unit. There will be a degree of surface pore filling in the boundary layers of the bed-face, which may exaggerate levels of water absorption from a free water surface, compared to that of a mortar bed. Surface pore filling will further influence results each time the unit is removed and returned to the test surface.

Referring back to Figure 7.2, the equation of the line of best fit, the gradient of which gives the sorptivity factor, passes through the origin. The equations of these lines together with the respective correlation coefficients, are as follows:-

| | | |
|---|---|---|
| Water surface | $M = 1.4224 \times t^{1/2}$ | $R^2 = 0.88$ |
| Mortar surface | $M = 0.557 \times t^{1/2}$ | $R^2 = 0.92$ |

The effect of not fixing the point of origin on the line of best fit causes the line to intercept the y-axis at a point above zero, corresponding to the level of water absorbed during the initial surface wetting and boundary pore filling. The equations and intercepts are given below:-

| | | |
|---|---|---|
| Water surface | $M = 1.202\ t^{1/2} + 1.35$ | $R^2 = 0.91$ |
| Mortar surface | $M = 0.54 \times t^{1/2} + 0.12$ | $R^2 = 0.92$ |

It can be observed that the purpose of not fixing the equation of the line to pass through the origin improves the correlation coefficient. The intercept for initial water absorption from the mortar bed is marginal, however the effect of surface wetting and boundary pore filling from the free water surface is noticeable. This demonstrates that water absorption from a mortar surface is less influenced by starting-up state, largely due to the brick not being submerged in water and there being limited excess water available at the suction face.

It has been reported that water absorption levels associated with water extraction from a mortar are concerned with instantaneous values, which are not necessarily consistent with overall sorptivity. Results are further confused by disruption to capillary rise for the purposes of measurement and the availability of excess water at the suction face, which complicates readings around the origin and thereby makes the determination of instantaneous values impractical.

If the assumptions of linearity and point of origin are accepted, then sorptivity for any particular unit can be determined from a single point measurement. Sorptivity theory therefore, does not provide any further information over the conventional Initial Rate of Suction test, an appraisal of which was made in Chapter 6 and found to have no correlation with bond strength.

In order to identify the true relationship between unit water absorption and time, a method is required which will permit continued and uninterrupted flow measurement.

## 7.3 Determination of Continuous Water Absorption Profiles

There exists a complexity when attempting to monitor continuous, uninterrupted water absorption of a unit with time, in that it is first necessary to isolate the brick from the water reservoir, in order to weigh the quantity of water transferred between a free water surface and the brick. Attempts to weigh the continuous quantity of water absorbed from a tank of water would not be possible because the water entering the unit would not be distinguishable from the water remaining in the tank. Furthermore, the submerged surface would displace water, resulting in a buoyancy effect, recorded on the balance. It is not possible to weigh the brick in isolation for the same reasons.

A method of suspending the submerged brick in water and weighing the volume of water displaced by the sample, which would reduce as water was absorbed into the sample, was considered. However, this method was deemed impractical for this study, since it was found that the sample could not be submerged sufficiently quickly to provide instantaneous readings.

The adopted method was discovered during trial experiments, which attempted to lower the brick bed-face gradually into a free water surface. It was observed that as the brick bed-face approached the water surface, to within a distance of approximately 3-mm, the surface energy of the sample broke the surface tension of the free water surface. This had the effect of causing water to move above the water level in the tank, attracted by the brick bed-face surface energy. This mechanism provided a means of isolating the brick from the tank of water to allow a continuous measurement to be made of the quantity of water uptake.

The magnitude of the surface tension attraction causes an increase to the weight of the sample; however, provided that the area of the water reservoir is sufficiently large, maintaining a constant distance between the water surface and the unit bed-face, the surface tension force remains constant. The curve of the continuous water absorption readings can be adjusted retrospectively by a quantity equal to the initial surface tension force. This value can be quantified by two independent means. Firstly, the surface tension force can be attained by removing the brick from the water surface, thereby breaking the surface tension. This causes a peak in the experimental water absorption curve, equal to the size of the surface tension force. An independent measurement of the final mass of water absorbed by the unit at the end of testing, determines the difference between the experimental results and the true quantity of water absorbed.

The phenomena of surface tension proves advantageous to this study since it provides a supply of water directly to the underside of the brick bed-face. Since the sample is not submerged beneath the water surface, the bed-face is not saturated with excess water. This approach allows for the immediate capillary uptake of water and surface filling of pores in the boundary layer of the bed-face in a similar manner to water absorption from a wet mortar.

Surface tension therefore provides a means of transferring water to the brick bed-face, in opposition to gravity, as the unit suction demands, rather than saturating the surface. The forces of surface tension possibly act in opposition to hydraulic suction, however it is probable that this method is more representative of the true condition of the state of the boundary layer between the brick and the mortar.

### 7.3.1 Methodology

The apparatus used to measure the continuous water absorption of a porous media from a free water surface is shown in Figure 7.3 below. The experimentation briefly comprises of the following apparatus:-

> A large reservoir of water; this must have plan dimensions sufficiently large enough to ensure that any reduction in water level resulting from sample absorption is negligible. A significant reduction in water level over the duration of testing could influence the magnitude of the measured surface tension force. A Perspex tank having approximate dimensions of 500mm x 300mm plan dimensions at the level of the water surface was used. The tank was positioned on a machined flat bed of the testing machine. A 3-mm thick glass plate was

then placed on top of the tank and levelled using a spirit level in perpendicular and diagonal orientations. This ensured that that the rim of the tank and the water surface remained parallel.

A means of suspending the sample parallel with the free water surface. The sample was supported by a purpose made aluminium collar which fitted over the top of the brick and held the brick by four self-tapping screws which were embedded into the stretcher face. Initial methods of supporting the unit utilised rubber pads fitted to the head of bolts, however it was observed that the brick could slip during testing. As an alternative, self-tapping screws embedded into the brick surface and held the sample steady during testing. The glass plate was placed on top of the water tank with the brick on top in order to ensure that the bed-face was parallel with the water surface. The cradle was then lowered over the brick and the locating screws tightened before removal of the glass plate.

A means of continuous measurement of the mass of water absorbed by the sample. A balance strain sensing load cell, with 3kg maximum load was removed from a balance and the electronic impulse (2V) was connected to a Solatron Data Logger, which produced a continuous signal output. This method was preferred to using a balance with an on screen display, since results could be continually recorded against time.

A means of controlling the rate of lowering of the unit towards the water surface. This was achieved using a Haynes Compression Testing rig, which had a regulated loading rate. The sample cradle was suspended by a 8-mm diameter threaded steel rod from the load cell which in turn was fixed to the loading ram of the loading rig.

Each sample was independently weighed in the dry state before testing and numbered with an indelible felt tip pen. The brick was then suspended parallel with the water surface as described above. The data logger was initialised which meant that the weight of the sample was not recorded by the data logger. The sample was then lowered at a constant rate towards the water surface until it was observed that there was a sudden jump of water to the underside of the brick. To aid the immediate determination of the contact point, a small mirror was positioned in the base of the tank at an angle of 45-degrees towards the operator and a spot-light illuminated the water. Immediately the surface tension jump occurred there was a flash of light from the mirror and the travel of the machine was stopped and a stop-watch started. During each test run the operator would monitor the bed-face zone to ensure that the surface tension zone was uniform across the bed-face.

Each test was run for a period of 120-seconds and then the brick was rapidly removed from the water surface by the reverse action of the machine under the fastest possible loading rate. Logging of the data continued for at least 10-seconds after removal of the sample in order to obtain a stable reading for the total weight of water absorbed by the sample over this period. The glass plate was returned over the top of the tank and the sample released. The brick was weighed by a separate balance in order to provide an independent confirmation of the mass of water absorbed. An equivalent quantity of water absorbed was then returned to the reservoir before each subsequent test run.

The data was logged every second and at the end of each series of experiments the results were transferred to Microsoft Excel format to facilitate further analysis.

### 7.3.2 Experimental Results

Figure 7.4 shows the experimental water absorption profile for a typical Fletton brick. The sharp peak in the experimental profile at 120-seconds shows the force, in grams, required to break the surface tension between the brick bed-face and the water surface. The figure demonstrates that the force is equal to the magnitude of the surface tension constant, which occurs at the origin and is present throughout the test. The final part of the profile shows the recorded mass of water absorbed by the unit, once the surface tension forces have been removed; this weight is independently confirmed by weighing the unit on a separate balance. For the particular brick chosen, it can be seen that the surface tension jump is of a magnitude of 50-grams and occurs instantaneously upon contact between the bed-face and the water.

Having determined the magnitude of the surface tension force, the curve can be adjusted to reflect the true cumulative mass inflow. Although the surface tension jump occurs instantaneously, the results show the value distributed over the first two data points, corresponding to the first two seconds of contact between the brick and the water surface. Consequently, the first two data points of the corrected curve show negative values and these points are omitted when fitting a theoretical curve profile. It was accepted that since water is transferred to the sample by surface tension, rather than by saturation, then the theoretical experimental profile could be permitted to pass through the origin of the graph, as considered in Section 7.2.5.

The corrected curve in Figure 7.4 shows the cumulative mass inflow in grams, with respect to time. The gradient of the curve reflects the mass flow rate. It is observed that the mass flow rate and consequently the cumulative mass inflow are greater during the initial part of the curve, reducing to a point where the curve becomes asymptotic.

Figure 7.5 shows an adjusted experimental profile for two similar Fletton units, which by conventional methods of determining water absorption characteristics, have the same value of sorptivity. However, using the method of continuous monitoring of water absorption with time, it can be seen that the two units, while absorbing the same quantity of water in 120-second period, have different overall absorption profiles. Unit No. 167 shows a rapid initial flow rate which contributes to transfer of more water during the early period of contact, followed by a subsequent reduction in flow rate during the later part of the curve. Unit No. 166 shows a more gradual suction profile. Conventional methods of classifying suction profiles using sorptivity, do not reflect such difference between units, partly due to the assumption of linearity and partly due to the fact that differences in mass absorbed are made less significant by multiplying by the square-root of time component.

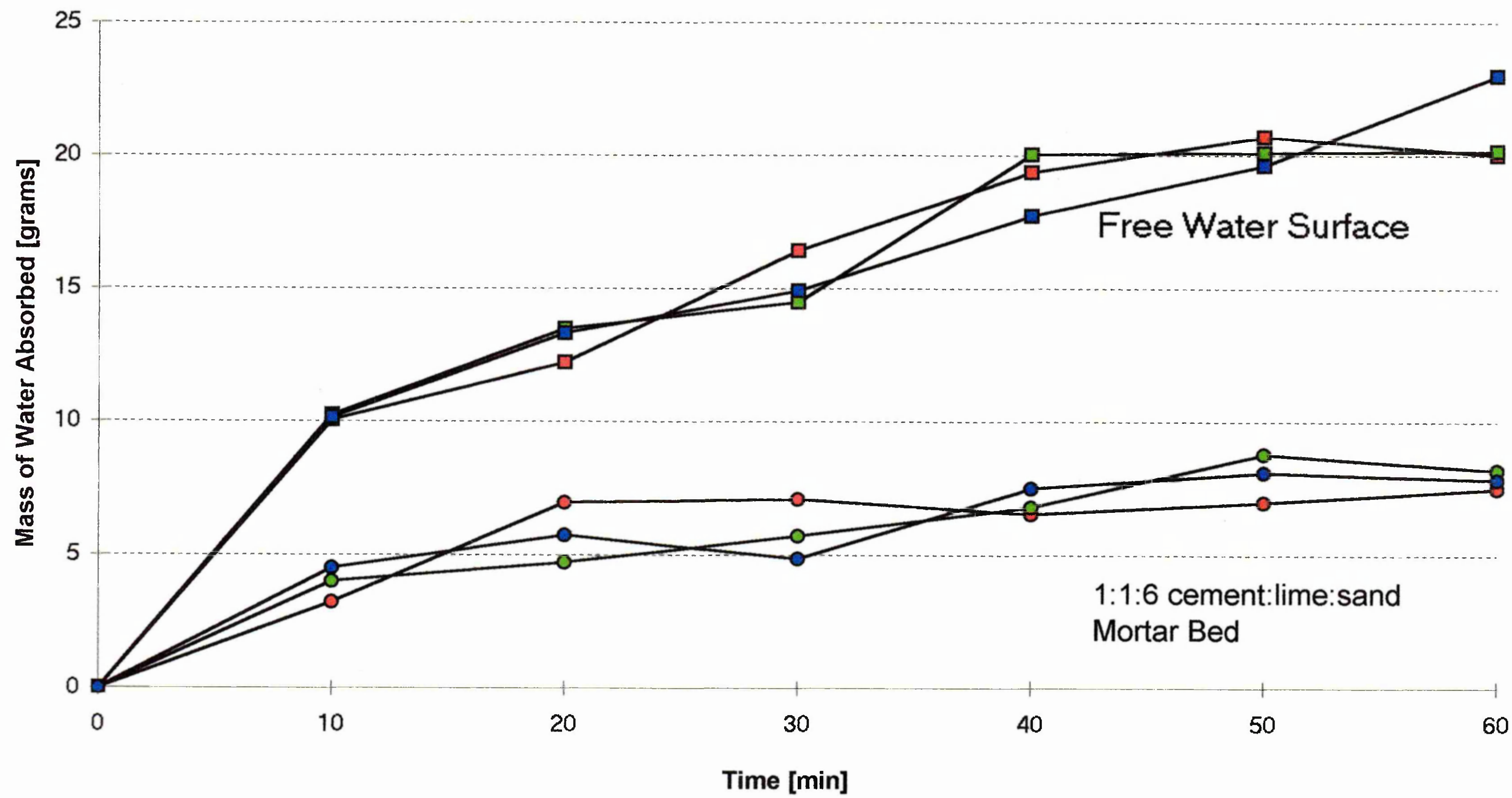



**Figure 7.1: Showing the Relationship Between Brick Water Absorption From a Free Water Surface and a Wet Mortar Bed- With Time**

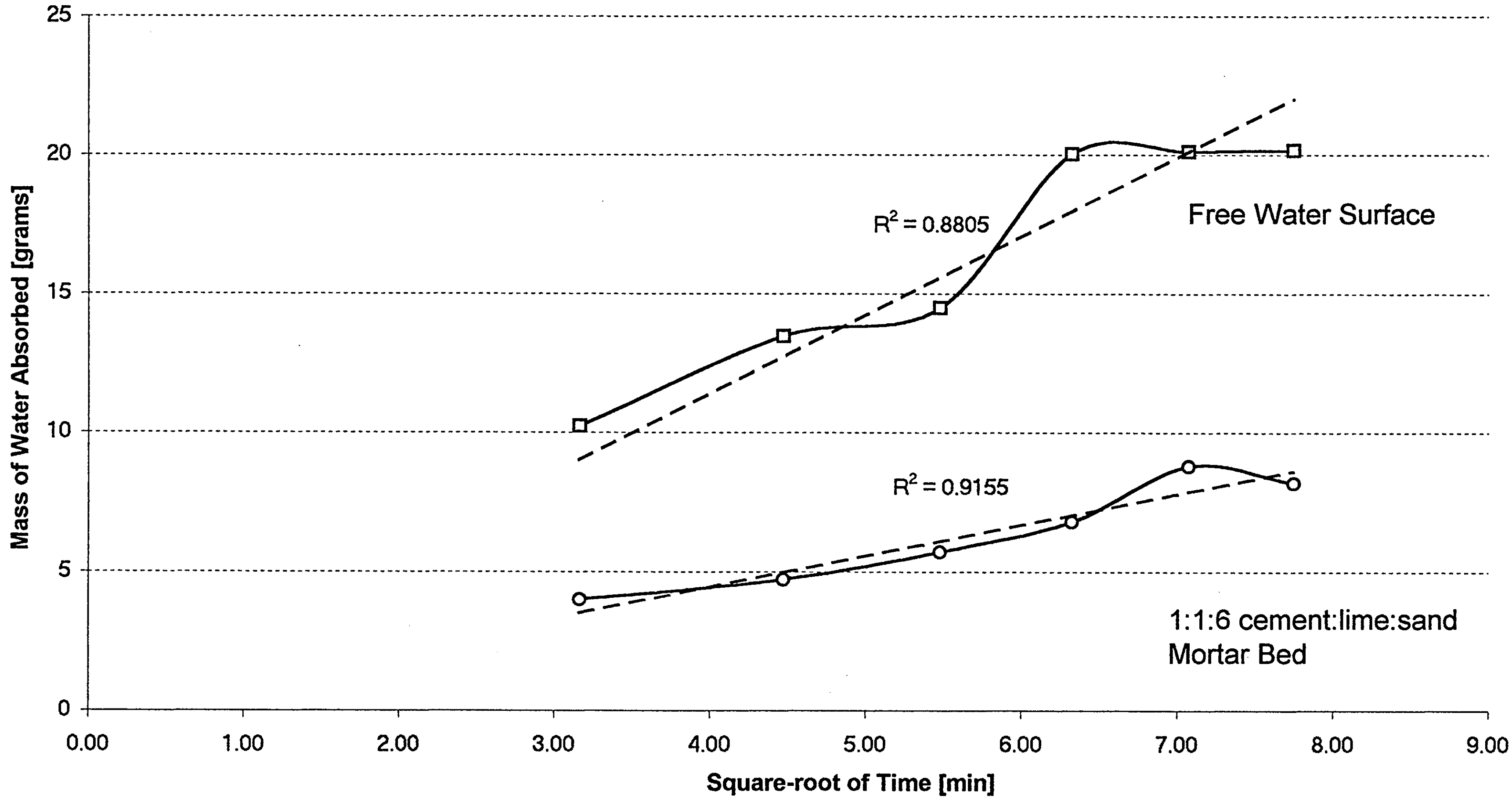



**Figure 7.2: Showing the Relationship Between Average Brick Water Absorption From a Free Water Surface and a Wet Mortar Bed- Against Square-root of Time**

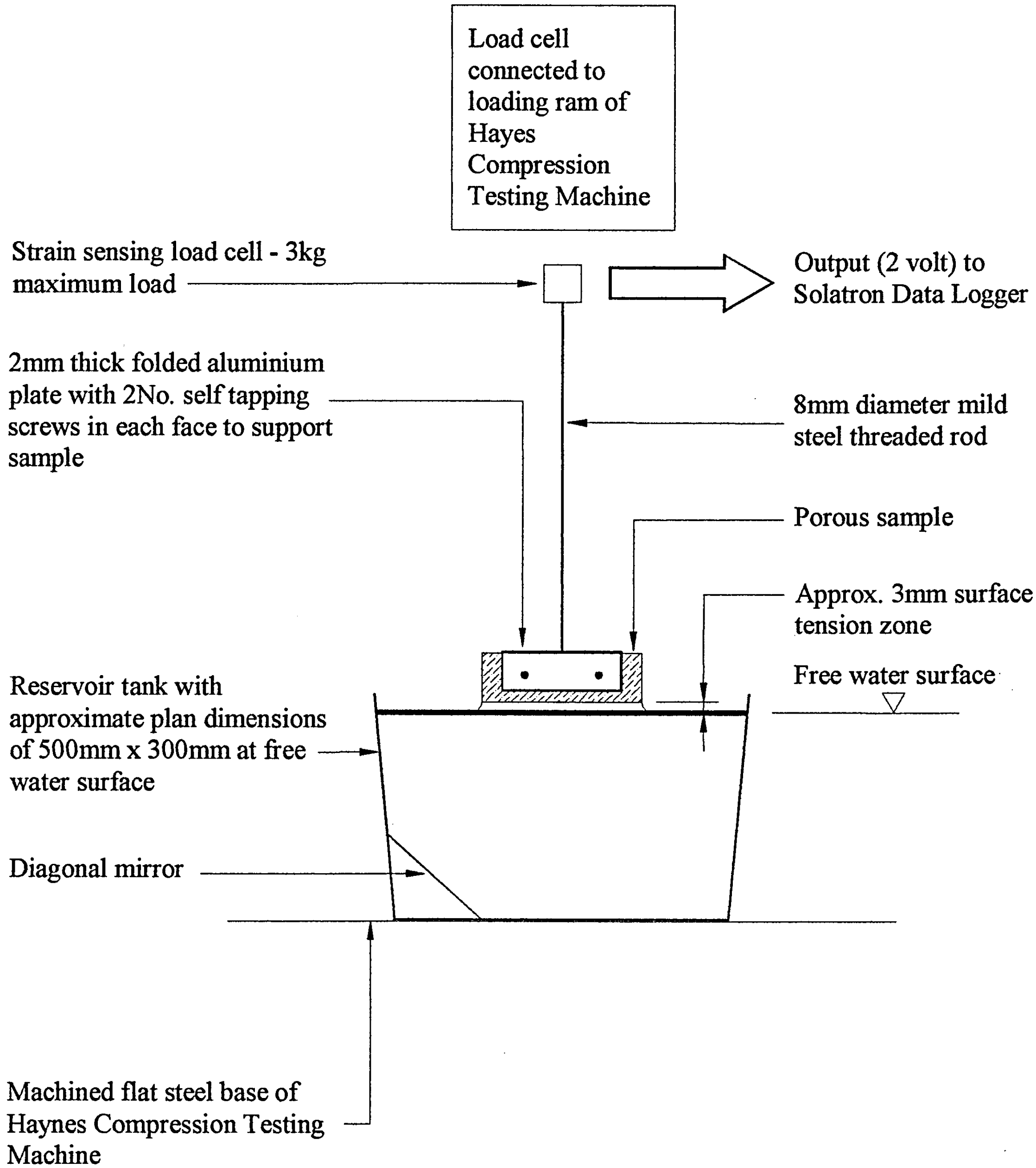



**Figure 7.3 :** **Showing Apparatus for Measuring Continuous Water Absorption of a Porous Material from a Free Water Surface** (not to scale)

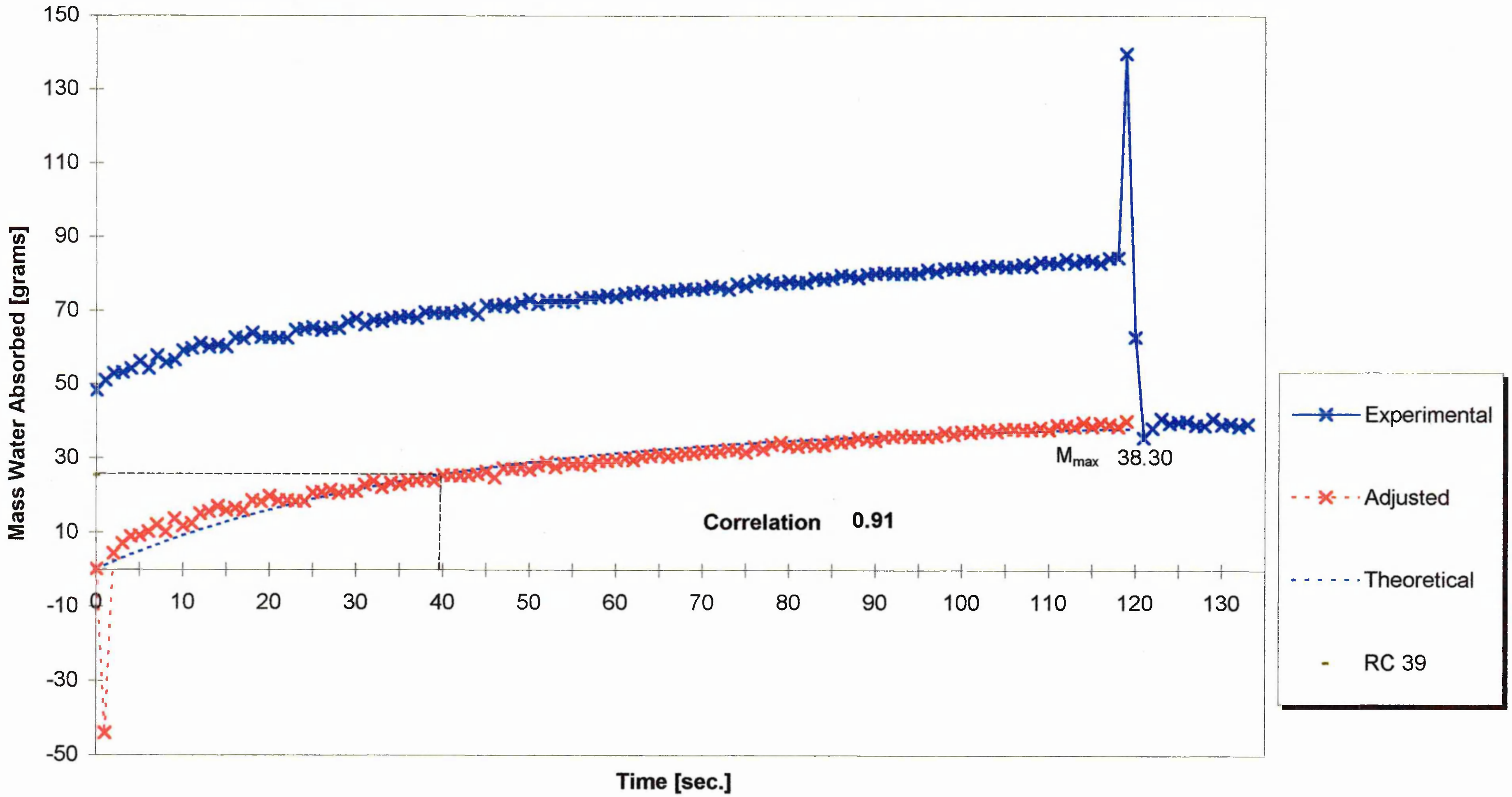



**Figure 7.4: Showing Both the Experimental and Theoretical Continuous Water Absorption Profile for a Fletton Unit**

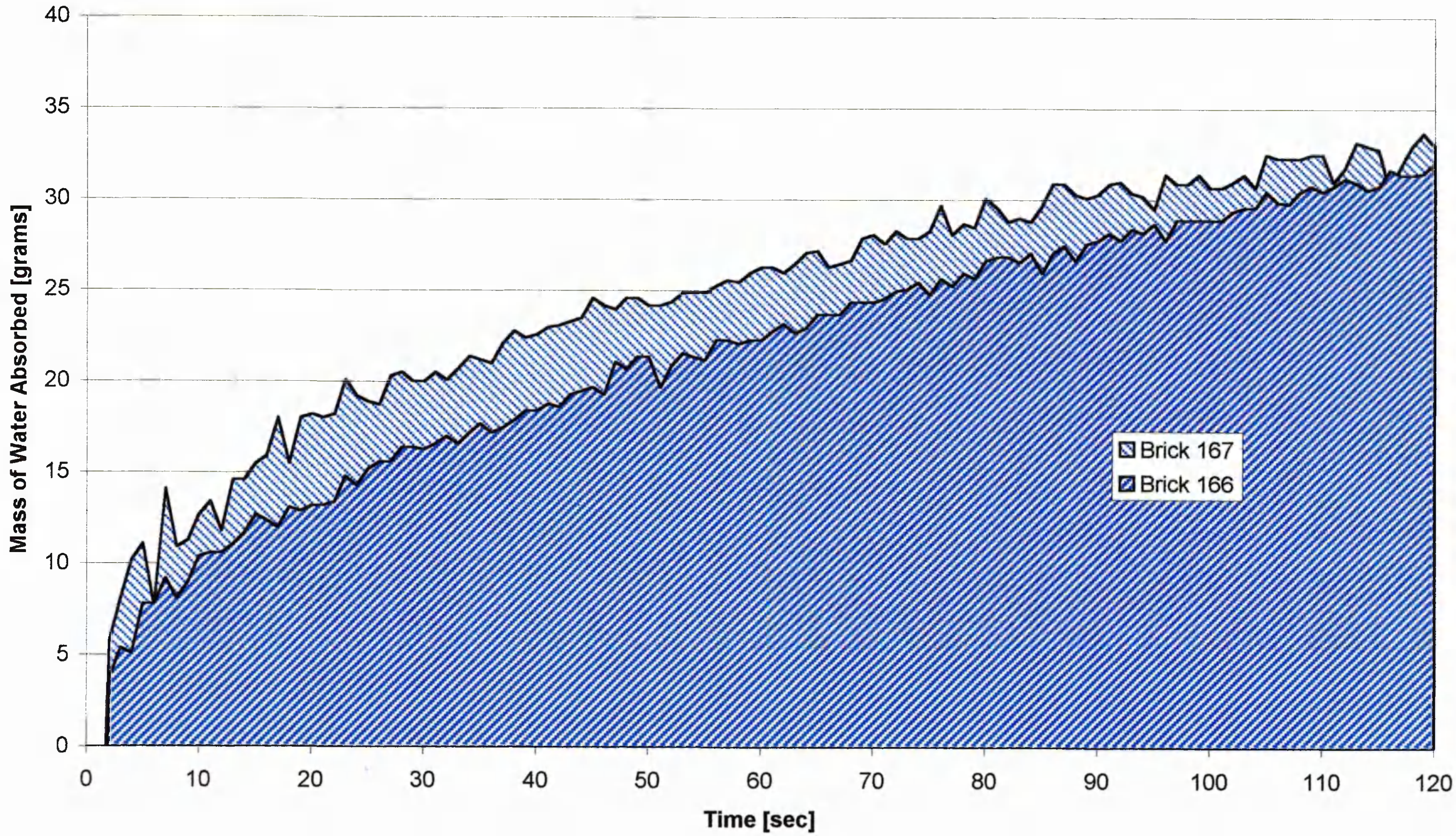



**Figure 7.5: Showing Difference in Continuous Water Absorption Profiles**

## 7.4 Classification of Continuous Suction Profiles

Having determined the shape of the continuous water absorption profile, it is desirable to fit an equation to the curve to enable the shape of the curve to be characterised in some way.

### 7.4.1 Consideration of Resistance Capacitance Theory

It was suggested in Section 7.0 that water inflow into a porous medium bears close similarities with many forms of exponential decay curves depicted by natural and physical phenomena. One such example is the charge held on the plate of a capacitor with time, the equation of which is shown below in Equation 7.2. A capacitor of C farads with V volts across its terminals has a charge of Q coulombs stored on one plate and –Q on the other.

$$Q = CV \qquad \text{Equation 7.2}$$

When considering capacitance in terms of water absorption capacity, the charge can be compared to cumulative mass of water stored by the sample and voltage compared to the mass flow rate. Likewise, the current I, may be considered as proportional to the rate off change of flow or the time derivative of flow. Application of capacitor rules gives Equation 7.3.

$$CdV/dt = I = -V/R \qquad \text{Equation 7.3}$$

If the voltage supplied by a battery is Vi, then the equation becomes:-

$$I = CdV/dt = (Vi - V)/R \qquad \text{Equation 7.4}$$

The above is a differential equation with the solution

$$V = Vi + Ae^{(-t/RC)} \qquad \text{Equation 7.5}$$

The constant A can be determined by the initial conditions: V = 0 at t = 0, therefore A = -Vi.

$$V = Vi\,(1 - e^{(-t/RC)}) \qquad \text{Equation 7.6}$$

The product of RC (resistance-capacitance) is termed the *time constant* of the circuit. For resistance in ohms and capacitance in farads, the product RC is in seconds. The profile of the above equation is shown in Figure 7.6. At the point on the curve where the time is equal to RC, the voltage is equal to 63.2% of the final voltage. Consequently, the profile of any curve obeying this equation can be determined from two values; the final value at the point where the curve reaches an asymptote and the time constant, RC.

### 7.4.2 Cumulative Mass Inflow With Time

Applying this theory to water flow into a porous body, the time constant RC can be likened to the capacitance of the pore volume and the resistance to flow of the interconnecting capillaries. Of importance to this study is that the definition of RC as a time constant, which describes the distribution of pore filling with respect to time, allows for a perception of the flow rate. Consequently, the water absorption profiles determined by the method of continually monitoring water uptake, can be classified by two variables; the final mass of water absorbed, termed here as $M_{max}$ and the distribution of this mass with time, given by the RC time constant. Modification of the resistance-capacitance equation shown in Equation 7.6 yields Equation 7.1

$$M = M_{max}\,(1 - e^{(-t/RC)}) \qquad \text{Equation 7.1}$$

Both characterising variables are dependent upon the determination of the final value of mass of water absorbed, which remains largely indeterminate given the potential long-term absorption capacity of a brick and the limited available water that can be extracted from the mortar. A porous brick unit for example, will continue to absorb water from a free water surface for a considerable period of time, up to the point where it reaches saturation. Levels of water absorption reported in Section 6.2.4 for the particular

Fletton unit under consideration, for 24-hour submersion, 5-hour boil and vacuum saturation, reach on average 393-grams, 442-grams and 423-grams respectively; this corresponds to a level of water absorption of between 21, 23 and 22%. The potential available water extraction from a fresh mortar bed is in the region of 12-grams, and consequently it is more appropriate to consider a finite region of pore filling, in the boundary layers of the brick bed-face. Referring back to Figure 7.4, it can be seen that the cumulative mass inflow curve becomes stable around the 120-second period and corresponds to a mass of water absorbed of between 30-grams to 60-grams. The brick will continue to absorb considerable amounts of water beyond this point, although the rate of absorption becomes negligible. Further justification of the acceptance of a finite zone of brick bed-face absorption is provided from Figure 7.1 which demonstrates that 50% of the excess mix water is removed within the first 10-seconds and that by 60-seconds the mass absorption has reached a steady state. It is proposed that unit suction forces, beyond 60-seconds, will not contribute to continued water abstraction of water from the fresh mortar, since the mortar will begin to establish capillarity of its own, exerting counter suction forces on the movement of water into the brick.

If for the purposes of curve fitting, it is accepted that the maximum level of water absorption for the suction profile is achieved by 120-seconds, then the value of the time constant RC can also be determined. This value was identified using the VLOOKUP function in Microsoft Excel spreadsheet programme to determine the time value from the adjusted experimental curve profile, which corresponds to 63.2% of the final 120-second value.

The derived value of RC, together with the final mass of water absorbed, $M_{max}$ enables the theoretical continuous water absorption curve profile to be plotted by substitution into Equation 7.1. The theoretical curve uses the origin as a point of reference and therefore provides a smooth curve profile, which helps to identify the rate of water absorption during the initial stage of water extraction. Figure 7.4 compares the adjusted experimental suction profile with the theory and identifies that excellent correlation can be attained.

The first and second derivative of Equation 7.1, shown in Equations 7.7 and 7.8, express the Mass Flow Rate and the Rate of Change of Mass Flow Rate respectively

$$\frac{dM}{dt}(M_{max}(1 - e^{(-t/RC)})) = \frac{M_{max}}{RC} e^{(-t/RC)} \quad \text{Equation 7.7}$$

$$\frac{d^2M}{dt^2}(M_{max}(1 - e^{(-t/RC)})) = \frac{e^{(-t/RC)}}{RC} \quad \text{Equation 7.8}$$

### 7.4.3 Consideration of Suction Force

Applying Newton's Second Law, the rate of change of momentum of a body is proportional to the force applied and takes place in the direction of action of that force. In fluid mechanics, the momentum of a particle or stream of particles is defined as the product of the mass and velocity. According to Newton's Third Law the fluid will exert an equal and opposite force on the solid boundary such as the mortar surface.

The application of fluid mechanics has only theoretical merit in this instance since flow will be affected by friction and surface tension forces. Notwithstanding, it is evident that mass flow rate is related to the suction force and the kinetic energy of the moving water.

Attempts to relate bond strength performance to the function of suction force are made complex by the fact that the force of suction may only influence the amount of water abstraction if the unit has the potential storage capacitance. Consequently the suction force and capacity of the unit must be interrelated and have complimentary characteristics. Attempts to relate bond strength to the particular water absorption profile need to consider both the time constant RC and the final quantity of water held in the boundary layer $M_{max}$.

Furthermore, although the second derivative of the derived expression for continuous water absorption with time, given in Equation 7.8 is a function of the suction force, this value is dependent upon the specific point in time that the tangent to the curve is taken. Consequently it is difficult to make a direct comparison between suction force and bond strength. It is therefore beneficial to consider the general profile of the water absorption curve and to determine a means of classifying each curve profile and its relationship to bond strength.

## 7.5 Characteristics of Some Unit Suction Profiles

An understanding of the way in which water moves into the porous media and how this may influence bond strength is best understood by examining general types of unit and unit treatment, which have readily identifiable bond strength performance characteristics.

### 7.5.1 Water Absorption Profiles of Suction Rate Adjusted Units

Chapter 6 demonstrated that suction adjusted units, of a certain docking regime, have a significant, but detrimental bond strength performance. This influence was not readily detected by measurement of water absorption using the standard IRS test. It is anticipated that the shape of the continuous water absorption profile may aid the identification of the mechanism that suction adjustment has upon the way water is absorbed by a unit.

A sample of 10 Fletton units were selected at random from the population of 200. Each sample was tested using the method of continuous absorption described in Section 7.3 and characterised by $M_{max}$ and RC. In order to ensure that the water absorption profile was unique to each individual unit the experiment was conducted twice. The samples were then suction rate adjusted using a regime of 2-minute submersion, followed by 10-minute drain with the bed-face uppermost as described in Section 6.2.3 and their water absorption profiles determined immediately after the drain period. Between each test, the units were dried back in an oven at 105°C, until constant weight was attained.

It was observed that the treatment of suction rate adjustment, by docking, had a different effect upon the water absorption profile, depending upon the particular unit.

In all cases, suction rate adjustment reduced the overall mass absorbed, $M_{max}$, by approximately half. This finding is in broad agreement with the work reported in Chapter 6.

In the majority of cases, the additional effect of suction rate adjustment was to increase the time constant RC. Generally, an increase in RC reflects a more gradual slope of the water absorption curve, accompanied by less marked initial absorption levels, as shown in Figure 7.7.

In some samples, the value of the time constant RC remained unchanged, reflecting little change to the profile of the initial part of the slope, accompanied by an early levelling out of the slope, as shown in Figure 7.8.

The results of each profile are summarised in Table 7.1 below, which show the parameters of $M_{max}$ and RC, necessary to categorise each profile.

**Table 7.1: Continuous Water Absorption Parameters, $M_{max}$ and RC for Suction Rate Adjusted and Non-Suction Rate Adjusted Fletton Units**

| | **Non-Suction Rate Adjusted Units** | | **Suction Rate Adjusted Units** | |
|---|---|---|---|---|
| **Sample No.** | **$M_{max}$ [grams]** | **RC [seconds]** | **$M_{max}$ [grams]** | **RC [seconds]** |
| 001 | – | – | – | – |
| 002 | 47.3 | 38 | 26.2 | 70 |
| 003 | 64.5 | 35 | 35.5 | 33 |
| 004 | 52.4 | 33 | 26.3 | 39 |
| 005 | 52.9 | 38 | 29.8 | 38 |
| 006 | 57 | 35 | 34 | 46 |
| 007 | 57.4 | 30 | 33 | 40 |
| 008 | 31.3 | 41.0 | 15.7 | 48 |
| 009 | 41 | 34 | 25.9 | 33 |
| 010 | 50 | 35 | 30 | 32 |

Suction rate adjustment of the units has the effect of filling the capillaries with water, thereby reducing the overall capacitance of the brick. Water will tend to fill the capillaries away from the bed-face due to the way the unit is placed during drainage. Saturation or partial saturation of the capillaries will result in reduced hydraulic suction forces. The influence that treatment by suction adjustment has upon the time constant RC will be dependent upon a number of factors. If the diffusivity of the unit is high, then water will be removed away from the bed-face so that there will be little influence upon the subsequent measured flow rate; only the quantity of water absorbed will be affected. If, however, the diffusivity is low, then water absorbed under docking will tend to remain closer to the bed-face boundary layer and will tend to reduce the rate of suction. This will be reflected by an increase in the time constant, RC. Of most significance, the overall effect of suction rate adjustment is to reduce the velocity of flow and therefore influence the suction force.

### 7.5.2 Water Absorption Profiles of Sliced Units

In order to model the effect of suction rate adjustment upon the water absorption profile, the brick bed-face was sliced from the body of the brick to give a 20-mm thick section with representative bed-face surface characteristics. It was anticipated that removal of the upper portion of the unit (which contained the frog), would reduce the capacitance but not necessarily influence the characteristics of the pores in the boundary bed-face layer.

The characteristics of the continuous water absorption profile for the sliced bricks shown in Figure 7.9 demonstrate close similarities to the suction rate adjusted samples shown in Figure 7.7 and 7.8. In general, the effect of slicing would appear to reduce, by almost one half, the overall capacitance of the region under consideration. It can be observed that the slope of the profiles for the sliced and whole units run approximately parallel beyond the initial part of the curve. The influence of slicing therefore, would appear to be to reduce the initial velocities of water flow in the first 5-seconds after contact.

There are several possible explanations for this effect. Firstly the removal of the upper portion of the brick involves removal of the frog; this dramatically reduces the surface area available for escape of air velocities through the upper surface as pores fill with water in the lower sections; the result is to retard the initial flow rate. In addition, the act of slicing the units could result in fine debris in the form of brick dust blocking the pores, again influencing the escape of air. Reduction in the capillary length could also reduce the potential hydraulic suction on water held in the surface layers of the bed-face pores, permitting premature drainage.

The bond strength performance characteristics of sliced units shall be reported in Section 7.7

### 7.5.3 Water Absorption Profiles of Pressed and Extruded Units

It is considered that particular types of brick exhibit typical water absorption profiles, dependent upon the nature of their capillarity, resulting from the particular manufacturing and firing processes. Water absorption profiles were determined for two types of pressed Fletton brick, a three perforated Kirton common brick and a Hepworth Class B Engineering brick. A particular type of Fletton brick was procured, which had been stacked in the kiln on the bed-face in order to leave an unblemished stretcher face. According to Whiteley[119], the kiss marks which appear where bricks have been placed in direct contact with each other during firing have a much coarser pore structure. The advantage of using the Fletton brick for this experimentation is that it has a plain bed-face area which is uninterrupted by perforations.

No attempt was made to investigate the water absorption profiles of aac block or calcium silicate units, because it was considered that diversifying away from clay units was contrary to the theme of this research programme.

In order to characterise water absorption profiles for different types of unit, a population of 20-replicates per population were measured for their continuous water absorption profiles by the method described in Section 7.3. Table 7.2 shows the water absorption parameters $M_{max}$ and RC for each of the four types of brick tested. Figure 7.10 shows the resulting theoretical water absorption profile for each sample, based on the average values shown in the table.

The modified Fletton units demonstrate a lower value of $M_{max}$ and an increased duration of RC when compared to the standard Fletton units used throughout the research programme. The prolonged time constant RC is indicative of the coarser pore structure of this type of unit; larger diameter capillaries exerting reduced hydraulic suction forces.

**Table 7.2: Water Absorption Parameters $M_{max}$ and RC for Different Types of Pressed and Extruded Units**

| | Standard Fletton | | Modified Fletton | | Kirton | | Engineering | |
|---|---|---|---|---|---|---|---|---|
| | $M_{max}$ | RC | $M_{max}$ | RC | $M_{max}$ | RC | $M_{max}$ | RC |
| **Mean** | 45.8 | 37.2 | 33.8 | 46.3 | 56.53 | 35.8 | 7.2 | 18.4 |
| **Standard Deviation** | 9.1 | 6.9 | 6.2 | 6.5 | 5.1 | 3.3 | 1.5 | 12.1 |

Mean and standard deviation based on 20-replicates for each sample population

The Kirton bricks demonstrate a high value of $M_{max}$ and a relatively short time constant, RC. The short time constant is indicative of the finer capillary nature of this type of extruded brick. Referring to Figure 7.10, it is apparent that the traditional Fletton unit and the Kirton unit exhibit very similar water absorption profiles and demonstrate a rapid initial rate of suction which if extrapolated, can be seen to approach equilibrium water content fairly rapidly.

The Engineering brick shows a very different profile due to its low water absorption. The rapid levelling-off of the continuous water absorption profile makes an accurate determination of RC difficult and it is possible that the shape of the water absorption profile for engineering bricks part conformity with the derived equation. The profile for the engineering unit demonstrates characteristics which may be beneficial to bond strength development by reducing long-term water extraction, however it is evident that whatever the suction force exerted by an engineering brick, the unit does not have sufficient capacitance to extract the excess mix water from the mortar bed. This in turn leads to excessive drying shrinkage of the mortar with the potential disruption to bond.

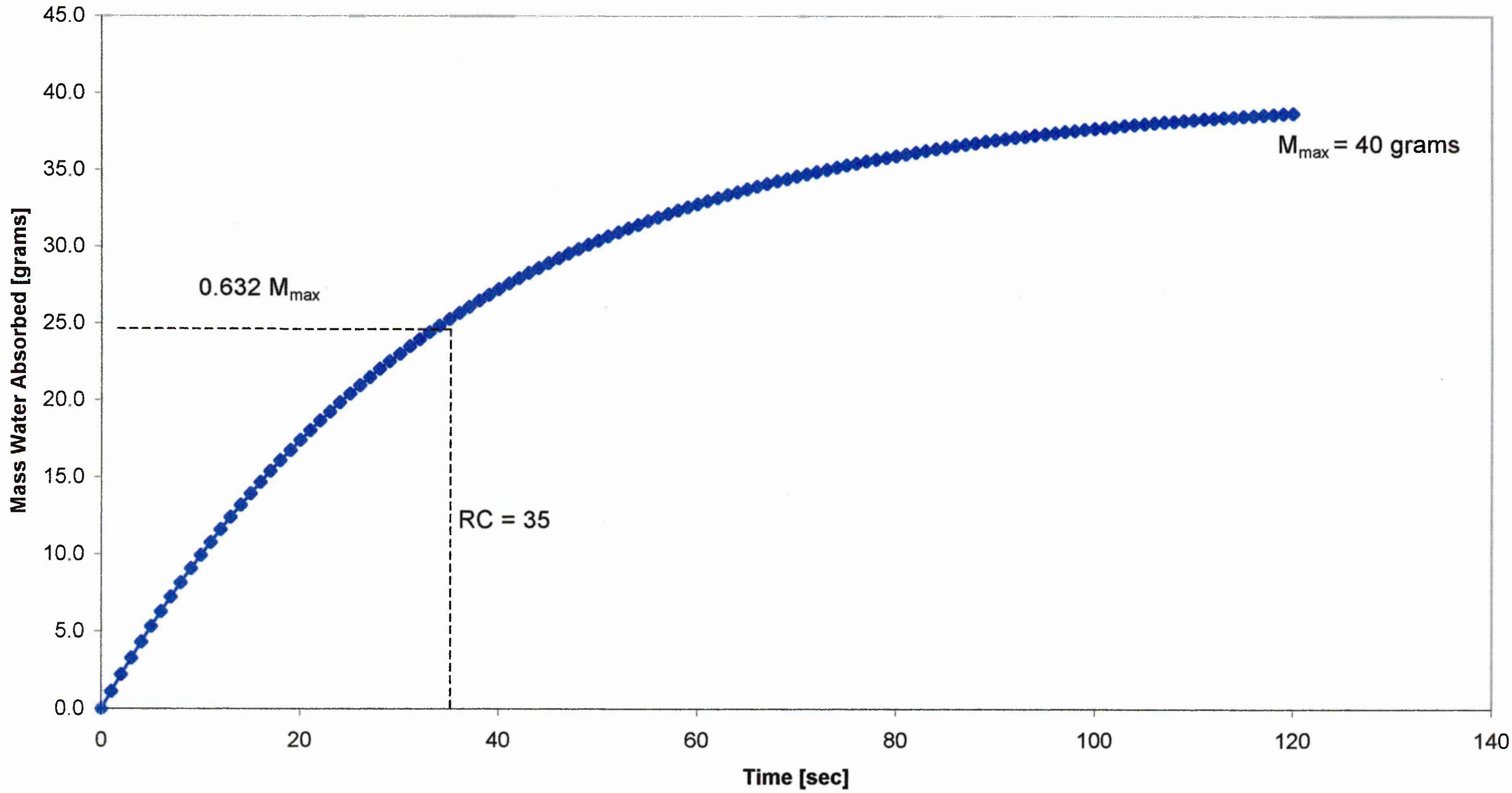



**Figure 7.6: Showing Theoretical Continuous Water Absorption Profile**

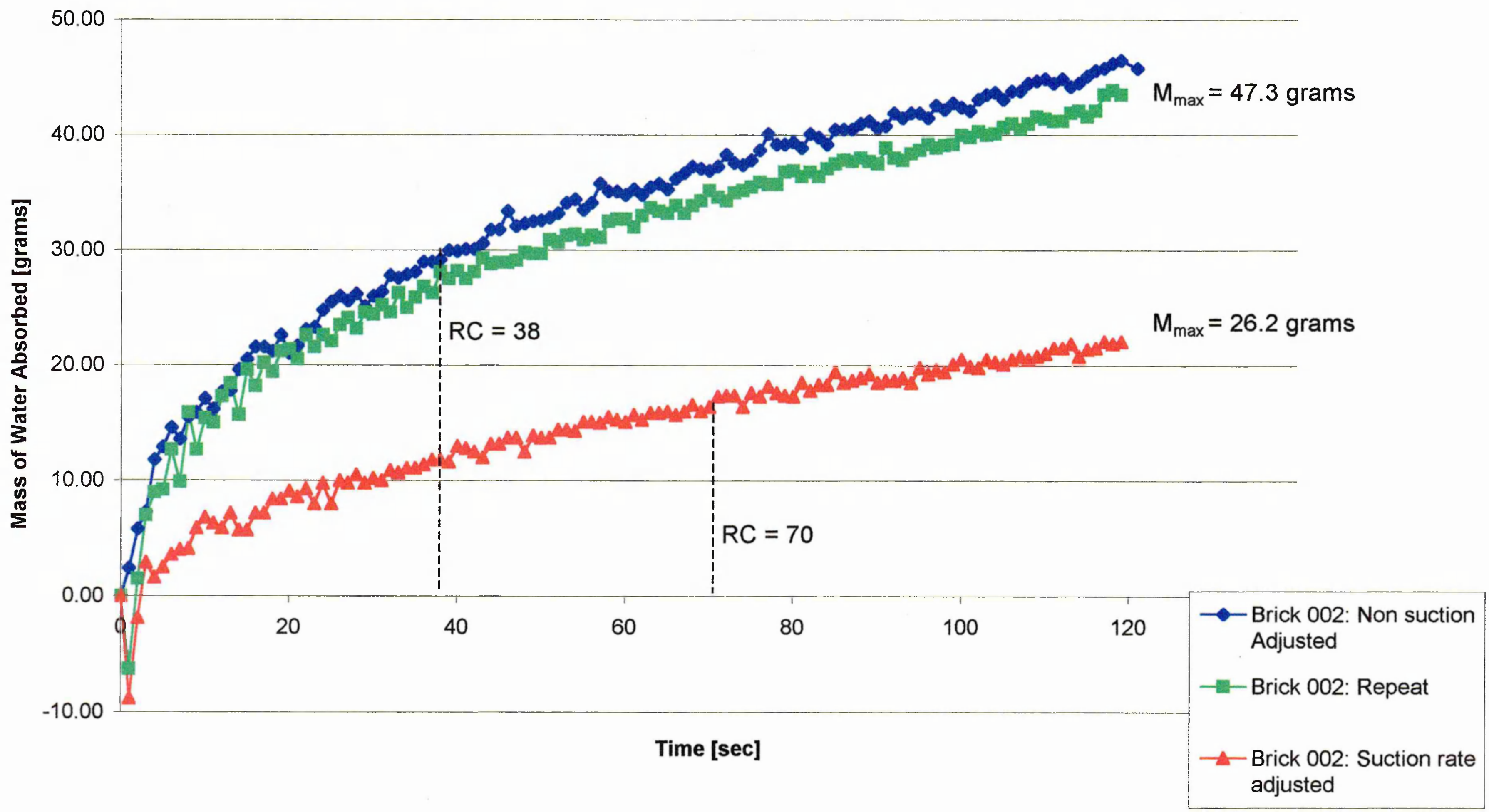



**Figure 7.7: Graph Showing Continuous Water Absorption Profile for Suction Rate Adjusted and Non Suction Adjusted Unit (002)**

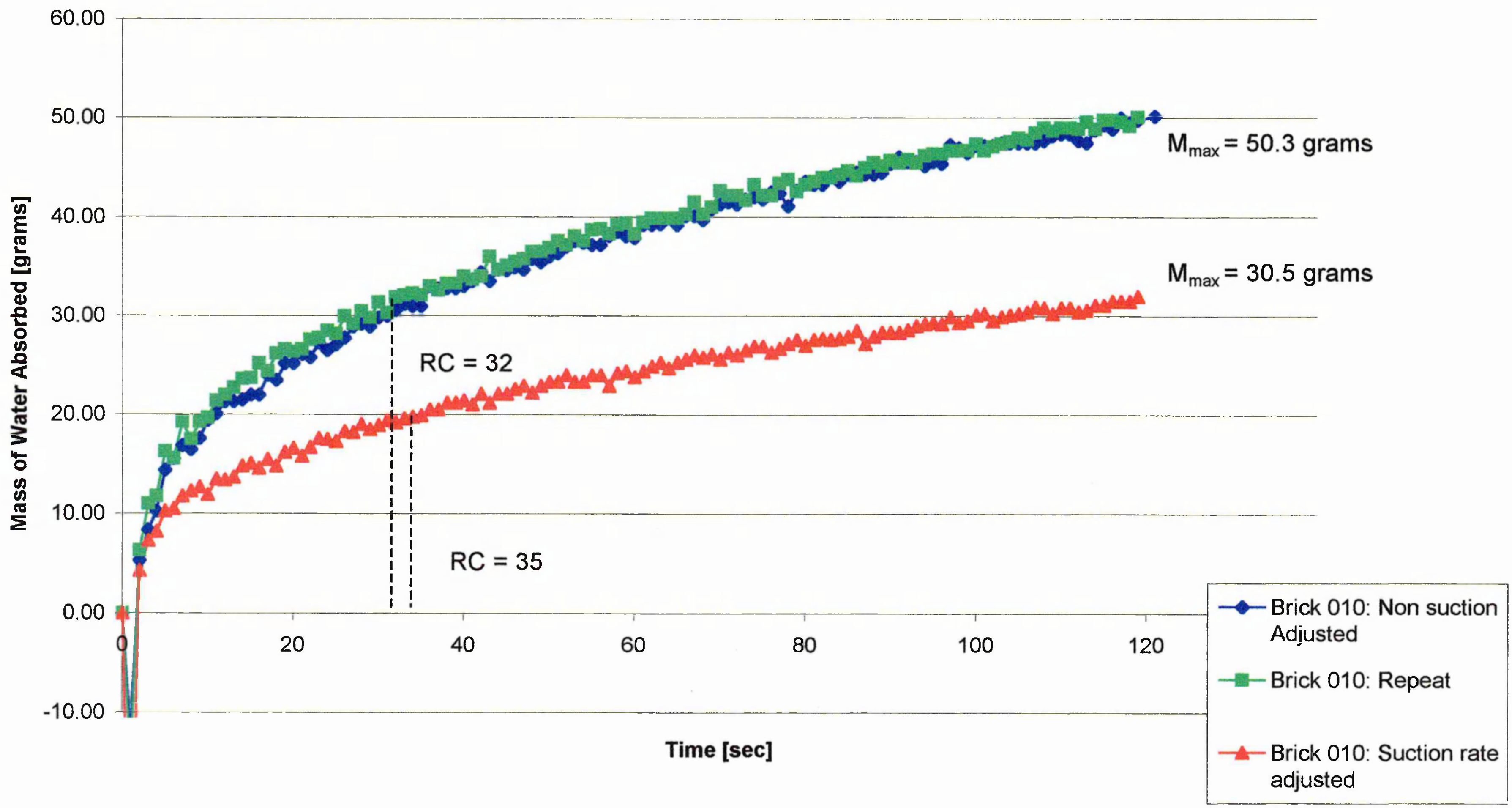



**Figure 7.8: Graph Showing Continuous Water Absorption Profile for Suction Rate Adjusted and Non Suction Adjusted Unit (010)**

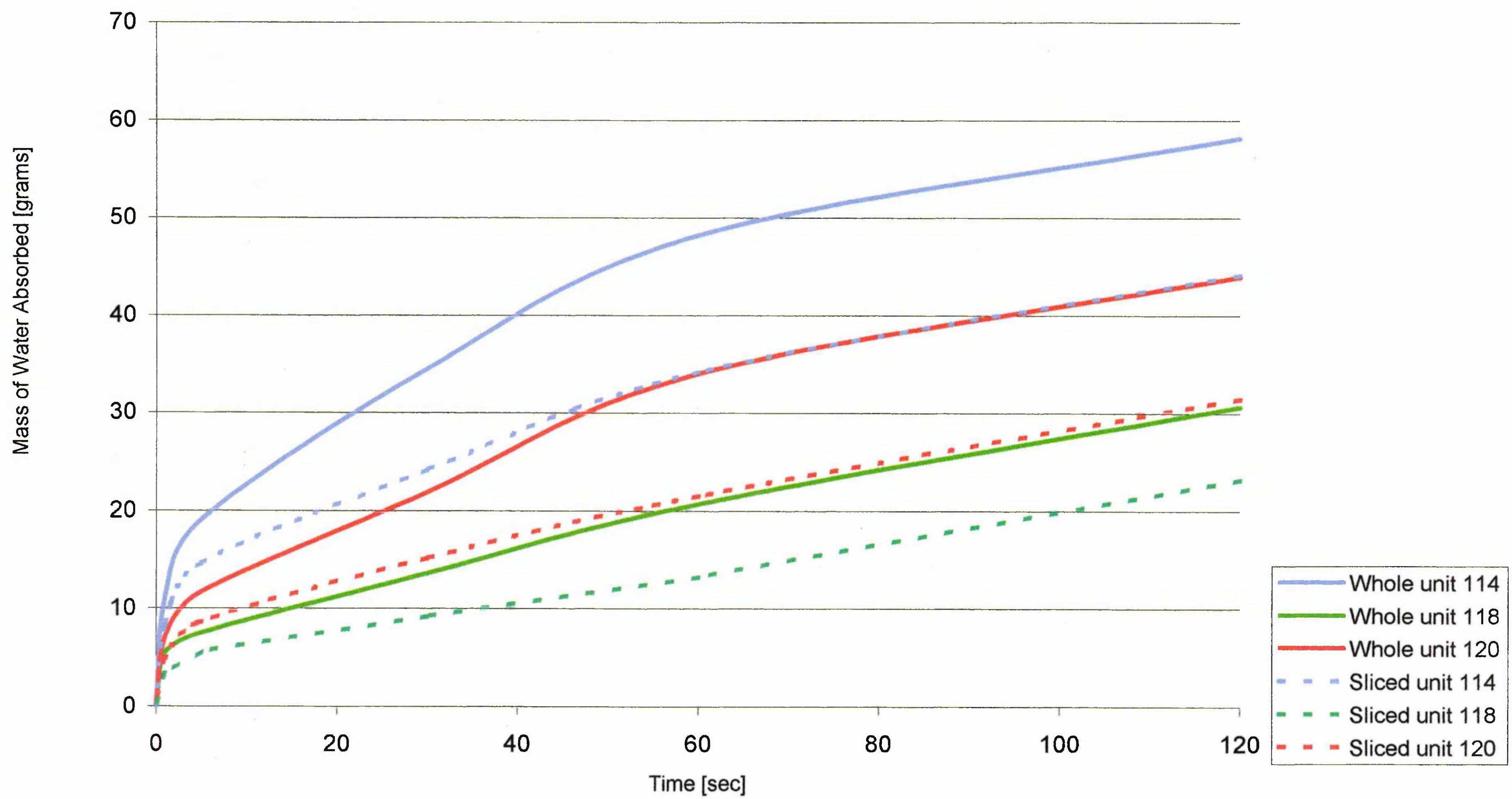



**Figure 7.9: Showing Relationship Between Continuous Water Absorption Profiles for Whole and 20-mm Sliced Flettons**

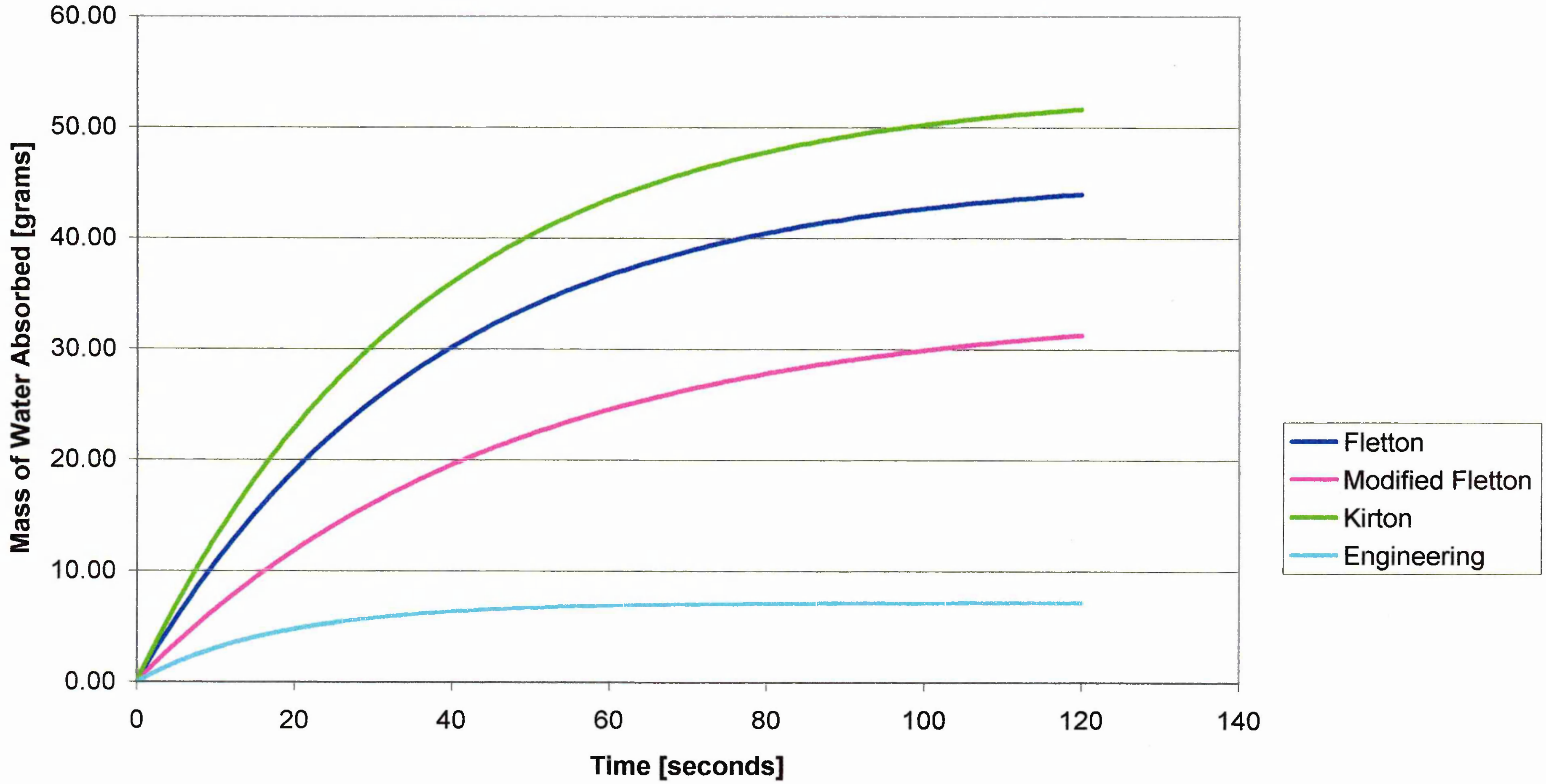



Figure 7.10: Showing Typical Theoretical Continuous Water Absorption Profiles for Different Types of Pressed and Extruded Bricks

## 7.6 Relationship Between Unit Suction Profiles and Bond Strength Performance Characteristics

### 7.6.1 Experimental Design

A sample of 200 Fletton bricks were tested by the method of continuous water absorption as described in Section 7.3. The corresponding values of $M_{max}$ and RC were identified for each sample number and samples ranked in increasing order of $M_{max}$. The samples were then separated into three distinct bands, corresponding to thirds of the percentile range, each containing 58-units. The mean and standard deviation of each tertiary was calculated to produce a sample range lying within 1±standard deviation of the mean. This gave three distinct groups of samples with similar 120-second absorption values, which differed significantly from their adjacent set.

Each of the three ranges were then ranked by increasing order of RC. Samples were then paired together using corresponding RC values. For the purposes of pairing it was considered that $M_{max}$ could differ by a few grams but that RC should not differ by more than 1-second. Each pair of bricks were taped with their bed-faces together and stacked in separate piles, one for each of the bands of lower, middle and upper tertiary.

The range of $M_{max}$ and RC for each band is shown below in Table 7.3.

**Table 7.3: Showing Range of Water Absorption Parameters $M_{max}$ and RC for Each Sample Population**

| | **Tertiary Range of $M_{max}$** | | |
|---|---|---|---|
| | **Lower** | **Middle** | **Upper** |
| **$M_{max}$ [g]** | 33.35 - 38.65 | 42.95 - 46.45 | 51.55 - 57.90 |
| **RC [sec]** | 31.5 - 43.0 | 29.5 - 50.5 | 29.0 - 39.0 |

Sample population ranked using $M_{max}$; Lower, Middle and Upper population represent tertiary ranges of average $M_{max}$ ± 1 standard deviation. Brick samples then paired using RC.

### 7.6.2 Couplet Manufacture and Testing

Each of the three central bands contained 14-couplets, which was considered to be the limit to the number of couplets which could be produced from any one mortar mix due both to the yield of the mix and the increasing spot-board age, as described in Chapter 3. In order to distribute any effect of mix properties across the three different sample ranges, couplets from each tertiary group of bricks were made in rotation using three separate mixes, denoted A, B and C. In addition, to counter any effect of increasing spot-board age, the order of manufacture was different for each of the three mixes, as follows:-

> Mix A: Samples taken from Lower, Middle and Upper bands in order of reducing RC value.
>
> Mix B: Samples taken from Lower, Middle and Upper bands in order of increasing RC value.
>
> Mix C: Samples taken from Lower, Middle and Upper bands. Samples taken from mid-range and manufactured by order of increasing RC to the top of the range and then by reducing RC to the bottom of range.

The mortar used for couplet manufacture was a 1:1:6 cement:lime:sand mix as described in Chapter 3.

Samples were then cured at 20°C and 80%RH for 28-days and then tested for bond strength in accordance with the procedure outlined in Chapter 2.

### 7.6.3 Manufacture of Couplets Using Sliced Units

During preliminary experimental work it was previously recognised that couplets manufactured using 20-mm thick sliced units demonstrated significantly reduced bond strength. The use of sliced units had initially been adopted for the purpose of weighing each unit after bonding, in order to attain whether there was any transfer of mortar fines to the body of the unit. Samples were sliced to enable their weight to be measured using a 500-gram balance, for increased resolution. The results of this work proved inconclusive and are not reported.

Notwithstanding, given the unusual water absorption profiles of sliced units, presented in Section 7.5.2, an analysis of bond strength for 28-day old couplets manufactured from sliced units are presented for comparative purposes. An assessment was made on these samples to ensure that the reduced bond strength was not a result of increased deflection during bond testing, due to the reduced section modulus of the material. The plotter output of travel against deflection showed a smooth profile, which would not have been achieved if there was tearing of the bond induced by increased deflection.

### 7.6.4 Experimental Results and Analysis

Table 7.4 shows average 28-day bond strength values for each group of samples. It should be noted that units selected for slicing were chosen at random and no water absorption parameters were measured for these units. The bond strength results for the sliced units are presented in the table for comparative purposes.

**Table 7.4: Average Tensile Bond Strength and Standard Deviation for Each of the Tertiary Ranges of $M_{max}$ and 20-mm Sliced Units**

| | Tensile Bond Strength [N/mm$^2$] | | | |
|---|---|---|---|---|
| | Tertiary Range of $M_{max}$ | | | 20mm Sliced Units |
| | Lower | Middle | Upper | |
| **Average** | 0.12 | 0.12 | 0.11 | 0.046 |
| **Standard deviation** | 0.03 | 0.03 | 0.03 | 0.009 |

The bond strength results were analysed using a Minitab statistical package and the following results obtained.

<u>A one-way analysis of variance (ANOVA) showed that there was no significant difference in bond strength values between each of the tertiary ranges of $M_{max}$. (ANOVA: $f=0.49$, $p>0.05$, $df=2,36$)</u>

The above statement shows that there is no discernible difference in bond strength between significantly different ranges of $M_{max}$. These results are in broad agreement with those obtained in Chapter 6, which concluded that the quantity of water absorbed is not indicative of bond strength performance.

However, the results did demonstrate that there was a significant difference between the bond strength of the sliced units when compared to those of the whole units for each of the tertiary ranges of $M_{max}$. It has been reported in Section 7.5.2 that the water absorption profiles of sliced units bear close similarities with the profile of suction rate adjusted units and consequently depict a reduced bond strength.

Figure 7.11 shows that there is a clear linear relationship between the time constant RC and bond strength, for each tertiary range of $M_{max}$. The regression demonstrate a reduction in bond strength as the time constant increases. The correlation coefficient by least squares analysis for each of the regression lines are shown in Figure 7.11 as 0.61, 0.93 and 0.68, for the upper, middle and lower tertiary ranges of $M_{max}$ respectively; however, there exists no such linear relationship between the time constant and bond strength across the full range of $M_{max}$.

### 7.6.5 Discussion of Results

The results show that there exists a distinct relationship between the tensile bond strength of a unit and the time constant of the continuous water absorption curve. The shorter the time constant, the faster the rate of water absorption and therefore the greater the suction force exerted on the mortar.

The relationship between the time constant and bond strength is only identifiable if the samples are divided into bands of $M_{max}$. No such relationship exists if a correlation between the time constant and bond strength is made across the full range of samples, suggesting that the parameters of RC and $M_{max}$ are mutually dependent. Furthermore, it is accepted that this relationship can only exist over the central range of results; for example, bond strength will not continue to increase indefinitely with a continued reduction in the time constant.

Studying Figure 7.11, there would appear to be a tentative trend indicating that time constants for upper range of $M_{max}$ are shorter generally than for the lower range of $M_{max}$. This is contrary to the understanding that a shorter time constant is beneficial to bond strength, since there exists no significant difference in bond strength between tertiary ranges. However, considering the resulting shapes of the absorption profiles, the overall effect of a high value of $M_{max}$ with a short time constant or a low value of $M_{max}$ with a prolonged time constant is to produce a curve profile which reaches equilibrium relatively quickly. This could indicate that it is not necessarily the rapid removal of the excess mix water upon initial contact between the brick and the mortar which is most beneficial to bond development, but rather that the sustained period of suction is reduced.

It is clearly beneficial for a unit to be capable of removing water from the mortar rapidly, since this encourages rapid consolidation of the mortar bed and the establishment of the lateral compressive forces on the bed-face, inducing the initial bond. This phenomenon was discussed in Chapter 4. A further advantage of rapid water removal is that the continued water absorption of the unit is reduced, thereby discouraging long-term drying shrinkage.

While it may be advantageous for a sample to have a rapid rate of water absorption, demonstrated by a low time constant, water cannot be abstracted from the mortar unless there is sufficient storage capacitance.

In theory, it should be possible to relate bond strength to the rate of change of mass flow rate, using Equation 7.8; enabling identification of a trend across the entire range of $M_{max}$. However, this would be dependent upon choosing a specific point of time along the water absorption curve, at which to calculate the suction force. Attempts to relate bond strength to the rate of change of mass flow rate at 5, 10, 20, 30 and 60 seconds revealed no conclusive correlation. In reality, it is unlikely that suction parameters favourable to bond strength could be identified in this way. It is considered here that the bond forms as a function of both the rapid removal of the excess mix water in the initial stages and the subsequent reduction in water absorption with time, depicted by suction profile reaching equilibrium.

In consequence, the only way to relate bond strength parameters to the water absorption characteristics of the unit is to consider the actual shape of the continuous water absorption profile. The parameters of $M_{max}$ and RC allow the curve to be characterised in this way. By grouping the values of $M_{max}$ together into three distinct ranges, the result of the time component can be more readily identified.

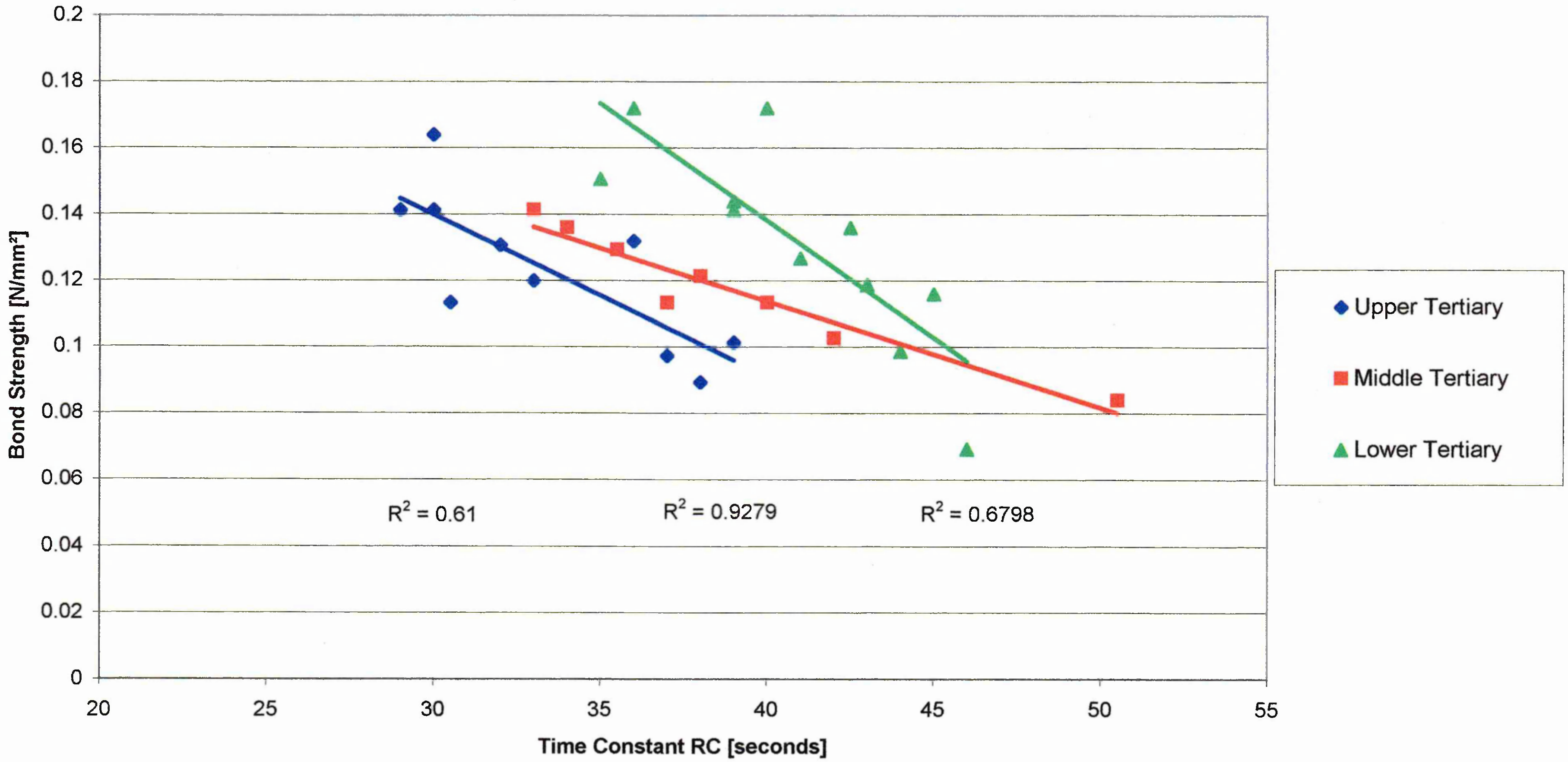



**Figure 7.11: Showing Relationship Between Time Constant and Bond Strength for the Upper, Middle and Lower Tertiary Bands Of Mmax**

## 7.7 Concluding Remarks

The experimental work reported in this chapter has revealed important information about the transfer of water between a free water surface and a porous material. Of most relevance to the study of bond strength development, the shape of the continuous water absorption curve describes the distribution of water absorption with time and allows for an assessment of the flow rate and suction force to be made. By using water absorption from a free water surface as an idealised model, an assessment can be made as to the potential suction force exerted on the mortar. Traditional methods of measuring the quantity of water absorbed from a free water surface over a specified time interval fail to identify a units potential water extraction from a retentive wet mortar bed. This explains why previous research has reported many conflicting findings between bond strength and the level of water absorbed by a unit.

An analogy has been made between the flow of water into a porous material and the behaviour of a simple resistance capacitance AC electrical circuit. This association remains conjectural, however application of the mathematical theory does allow the shape of the continuous water absorption curve to be characterised. Potentially, the water absorption parameters of $M_{max}$ and RC could be related to capillary structure within the brick. The component of RC could be further divided into the components of resistance and capacitance of the pore structure. However, it is considered that for the purposes of identifying bond strength parameters it remains sufficient to be able to describe the shape of the continuous water absorption profile.

The concept of suction force could help to identify the mechanism by which preferential failure planes are induced within samples manufactured from certain types of unit. Potentially suction forces exerted by the upper brick are greater where suction occurs under capillary rise, in opposition to gravity. On the contrary, suction exerted by the lower brick operates under conditions of partial saturated flow due to the initial infiltration of water leaving the wet mortar bed. This may help to explain why the vast majority of samples tested throughout this research programme demonstrated lower interface failure. Low absorption samples such as glass plate and engineering bricks tend to fail predominantly at the upper interface since, in this instance, more water is extracted by the lower unit.

It remains impractical to classify every masonry unit by the method described in this chapter. However it has been determined that it is possible to characterise the water absorption profile of different types of units and indeed units of the same type, which due to their location or stacking within the kiln during firing, exhibit very different bed-face pore structure.

It was discussed in Chapter 6 that the title of the Initial Rate of Suction test does not describe the true function of measurement of the test. However, it would appear that the act of suction rate adjustment, by docking, does indeed describe its function. Docking of units, which may be modelled by slicing of the units, reduces the capacitance of the sample and thereby limits the potential quantity of water abstraction. However, the magnitude of water absorption is rarely reduced sufficiently enough to have any limiting effect upon the quantity of water which can be abstracted from the wet mortar. It would appear therefore, that suction adjustment reduces the initial suction force, which would explain why the method is favoured by bricklayers, in order to promote the laying performance of the units.

## References

66. British Standards Institution. Specification for Clay Bricks. BS3921. 1985.

70. De Vitis N., Page A.W., Lawrence S.J. Influence of Age on Masonry Bond Strength: A Preliminary Study. Masonry International, Vol. 12, No.2, 1998.

86. Gummerson, R.J. Hall, C. Hoff, W.D. Capillary Water Transport in Masonry Structures; Building Construction Applications of Darcy's Law. Chartered Institute of Building, Vol.1, No.1, Paper 2 p17, 1980.

87. Morgan, J.W. Brick Absorption. Architectural Science Review. pp 66-68, 1977.

89. Murray, I.H. The Effect of the Absorption Characteristics of the Brickwork Background on the Adhesion of Renderings. BRE, Scot Lab.

90. West, H.W.H. A Note on the Suction Rate Test for Bricks. Proceedings of British Ceramic Society. No. 24, pp.21-31. Sept. 1975.

91. Anderegg, F.O. The Effect of Brick Absorption Characteristics Upon Mortar Properties. Proceedings of the ASTM, vol 42, p821. 1942.

103. Harbert, F.C. Moisture Measurement in Industry. Research, May 1971.

104. Kirkham, D., Powers, W.L. Advanced Soil Physics. Wiley-Interscience, New York, 1972.

105. Bomberg, M. Water Flow Through Porous Materials. Division of Building Technology, Lund Institute of Technology, 1971-2, Reports Nos. 19, 20, 21.

106. Schwarz, B. Capillary Water Absorption of Building Materials. BRE, Nov. 1972.

107. Kunzel, H., Schwarz, B. Die Feuchtigkeitsaufnahme von Baustoffen bei Beregnung. Ber. Bauforschung. Heft51. Berlin 1968.

108. Stull, R.T., Johnson, P.V. Some Properties of the Pore System in Bricks and Their Relation to Frost Action. Journal of Research, Nat. Bureau Standards, vol.25, p711. 1940.

109. Jansson, I. Testing the Rate of Water Absorption. IBID

110. Sneck, T. The Influence of Suction on the Mortar in the Joint. Proc. RILEM/CIB Symposium, Moisture Problems in Buildings. Helsinki, 1965.

111. Sneck, T. Dependence of Masonry Properties on the Interaction Between Masonry Units and Mortar. Proc. North American Masonry Conference. Boulder, Colorado, 1978.

112. Hall, C. Water Sorptivity of Mortars and Concrete: a review. Magazine of Concrete Research 41 (147) pp51-61. 1989.

113. Khatib, J.M. Mangat, P.S. Absorption Characteristics of Concrete as a Function of Location Relative to Casting Position. Cement and Concrete Research, Vol. 25, No.5, pp 999-1010,1995.

114. British Standards Institution. Methods of Testing Concrete: Methods of Testing Hardened Concrete for Other than Strength. BS 1881:Part 5: London 1970.

115. Levitt, M. Non-destructive Testing of Concrete by the Initial Surface Absorption Method. Proc. Symp. On Non-destructive Testing of Concrete and Timber, London 11-12 June 1969. Institution of Civil Engineers. London, 1970. Pp 23-28.

116. Sabir, B.B., Wild, S. and O'Farrell, M. A Water Sorptivity Test for Mortar and Concrete.

Materials and Structures, Vol.31, October 1998, pp.568-574.

117. Parrott, L.J. Water Absorption in Cover Concrete. Materials and Structures, 25, 284-292,1992.

118. Winslow D.N., Kilgour, L.K., Crooks R.W. Predicting the Durability of Bricks. Journal of Testing and Evaluation, Vol.16, No.6, November 1988

119. Building Research Establishment. Difficulties In Painting Fletton Bricks. BRE Information Paper IP22/79, 1979.

120. Ritchie,T. Measurement of Laminations in Brick. National Research Council of Canada, 1975.

121. May, J.O., Butterworth, B. Studies of Pore Size Distribution; The Effect of Firing Temperature. Reprinted from the Proceedings of Science of Ceramics Vol. 1 BCS,1961.

# CONCLUSION

A study of the research available in the literature on the subject of tensile bond strength of brickwork has identified considerable disparity with regard to those parameters considered to contribute to bond strength development. Despite significant publications on the subject of tensile bond strength of masonry, very few workers have presented theories regarding the nature of the bond development between the brick and the mortar. Consequently, the mechanism of bond strength development has remained largely an enigma.

It remains the author's view that much of the uncertainty regarding the nature of bond strength development is attributable to the particular testing methodology adopted to quantify bond strength. It is maintained that there exists a distinct difference between the nature of the bond strength measured by the application of flexural bending with that of direct tensile testing.

While it is recognised that the structural performance of the brick-mortar joint is best predicted by the application of flexural bending tests, it is argued that the actual bond strength between the brick and the mortar at the interface may only be quantified by direct tensile testing.

Appraisal of the direct tensile testing methodology used in this study and reported in Chapter 2, has demonstrated that The Sheffield Hallam University Tensile Test is capable of detecting, with significant statistical confidence, discrete changes in bond strength, attributable to various treatment types.

The application of flexural bending tests, for reasons explained in Chapter 2, has resulted in much of the research work drawing comparison between the development of the compressive strength of the mortar with that of the bond strength. This has further compounded the belief that the cement content of the mortar contributes in some way to the ultimate bond strength. In consequence, the result has been to establish a criteria for testing bond strength at 28-days, in accordance with the established compressive strength development of the mortar.

Experimental work conducted in Chapter 3, has shown that the cement content of the mortar does not appear to have any particular significance in relation to bond strength development, provided that the combined proportion of cement or cement and lime, maintain a 1:3 ratio with the sand. The traditional approach to mortar batching, using a proportion of cementitious material equal to approximately one-third the volume of the sand, has been reinforced by this work.

It is considered that the use of the constant mass sand batching methodology has aided the identification of the contribution played by cement and lime in the bond formation process. Previous studies, which have considered the influence of mortar volume ratios upon bond strength, have only done so against a multivariate background. It is argued that the approach of using a constant mass of sand should be used as a framework for the batching of laboratory mixes used for comparative purposes. A further outcome of applying this methodology is that it has been demonstrated that the volume of mix water should roughly match the volume of the cementitious material. One practical implication of this finding is that the addition of mix water on site, required to produce a given workability, could be specified at the design stage, provided that the batching is carried out by volume and that the quantity of cementitious material known for each batch.

The outcome of the observation that cement content of the mortar is not directly related to bond strength has prompted the investigation of how the bond strength develops with time.

The early bond strength development curve, derived from the results in Chapter 4, has shown that the bond is initialised within the first few seconds of contact between the brick and the mortar. From that point, the bond develops at a considerable rate, far in excess of the strength gain that could be attributed to the process of cement hydration alone.

Furthermore, results from Chapter 5 have shown that bond strengths can actually decline over time and that the direction of strength gain, past the 28-day benchmark for testing, remains conjectural. Results have shown that up to 40% of bonds can fail after 2-years, due to the influence of sustained long-term drying shrinkage, which if excessive can potentially rupture the mechanical bond.

The phenomena that the bond strength mechanism may be disrupted over time is highly significant when considering the strength, integrity and durability of the finished brickwork. Such potential shrinkage failures may be masked in the finished wall, but may give rise to planes of weakness in flexural strength or permit isolated water penetration between the brick-mortar interface.

Unfortunately, 28-day characteristic flexural strength values given in BS 5628, or alternative forms of bond testing carried out at this age, may not readily identify such potential disruption to the future bond. Consequently the value of using 28-day strength tests as a means of predicting both structural and durability performance of masonry is challenged by this work.

The shrinkage theory presented in Chapter 4 and Chapter 5 considers bond strength development as a mechanical process, with bond driven not by forces normal to the bedface, but rather parallel to it. It is presented that volumetric plastic shrinkage of the mortar bed, induced by rapid removal of the excess mix water by brick background suction, generates a mechanical lateral gripping action to the undulations of the brick bedface as the bond develops. Unlike alternative explanations of the bond formation mechanism, shrinkage theory may explain the rapid initialisation of the bond and the abrupt failure of the bond at mature ages. Shrinkage is one process which continues over the bond development period.

Early plastic shrinkage and long term drying shrinkage are considered to be synchronous with unit water absorption characteristics. This study has identified that the critical parameter controlling bond strength development is the removal of excess mix water from the mortar bed, within the first few seconds of the bond formation. The instantaneous suction force, which occurs upon immediate contact between the brick and the wet mortar, contributes to the critical removal of excess mix water by brick suction forces. If suction forces are high, depicted by a steep slope on the continuous water absorption curve, then rapid plastic shrinkage of the mortar will ensue. If on the other hand the force of suction is diminished, then less of the excess mix water will be removed from the mortar, leading to protracted levels of drying shrinkage at the brick-mortar interface.

It is concluded that the Initial Rate of Suction test is not sufficiently representative of a unit's ability to withdraw water from a retentive mortar bed. The practice of suction rate adjustment by docking has been shown to be detrimental to bond strength performance. Suction rate adjustment not only effects the degree of water uptake, but also alters the way water is drawn from the wet mortar bed into the brick, by diminishing a unit's potential suction force. Notwithstanding, it is recognised that workmanship plays a key role in promoting good, well-bonded, watertight masonry and in consequence, the loss in potential bond may be compensated for by improved workmanship.

This thesis maintains that the quality and nature of the contact at the interface between the brick and the mortar, characterised by the magnitude of the measured bond strength, is not only reflective of the integrity of the joint, but remains indicative of the compatibility of the component materials. Compatibility between the properties of the brick and those of the mortar not only affect the future performance of brickwork, but also promote construction productivity and beneficial bricklaying practices. Component compatibility is essential if brickwork is to be promoted as a watertight composite material, capable of performing the durability requirements induced by the modern nature of the building fabric.

In response to the identified weakness in the present knowledge base, this research programme has provided the opportunity to study more closely those areas of ambiguity which have arisen largely due to the application of a flexural bending testing approach.

It is considered that a significant contribution to the present state of the knowledge on the formation of the tensile bond strength has been made. In part this contribution has stemmed from the questioning of existing theories and practices but has also been made possible by the application of unique methodology, both for the experimental design and the approach to tensile testing.

# ADDENDUM

## Chapter Summary

This chapter has been included as an addendum to the thesis and incorporates a review of the most recent literature cited on the subject of masonry bond strength. The review examines work published during the period between the end of the research programme and final submission of the thesis.

# 9.0 Introduction

This final review of literature provides a synopsis of the more contemporary research work on the subject of tensile bond strength published during the last decade.

The literature review has identified specific areas of study which have been the focus of research. Advances in microscopy techniques have allowed for a much more informed understanding of the microstructure and constituents formed at the bonding interface. Microscopy has also aided more sophisticated studies of water transport through the mortar and across the bonding interface resulting from brick background suction. There has also been greater emphasis placed on the study of masonry cement mortars and their influence upon tensile bond strength.

Insight into the phenomena of mortar brick bond, until now, has been based almost exclusively on the study of bond performance and material characteristics after hardening of the mortar bed-joint. Very little is known about the effects of water flow in the mortar after brick laying, whereas the influence of these effects on the hydration conditions and mortar composition in the interface zone are assumed to be considerable.

The following review of literature provides an insight into the processes which occur during joint formation and examines their relevance to the findings of the current study.

# 9.1 Discussion of Literature

### 9.1.1 Water Transfer

Perhaps the most recent comprehensive investigation into the effect of water flow on mortar-brick bond has been undertaken by Groot[122] in the Netherlands. Groot recognises that water transfer from fresh mortar to brick may cause changes in material composition and modification of water distribution over the bed-joint cross section.

Groot acknowledges that mortars with high compressive strength do not necessarily demonstrate good bond strength performance. Groot explains that hardening or curing conditions in the body of the mortar joint may differ considerably from conditions at the bonding interface. He suggests that the transport of fine particles of the binding agent towards and across the brick-mortar interface, by brick background suction, have significant influence on the micro-structure at the bonding interface. Consequently, Groot's findings are in broad agreement with this piece of research, which has considered that the study of flow velocities of the mix water from the mortar to the brick are important.

In a recently published paper, Groot[123] describes the use of neutron radiography to measure water velocities during the first minutes of mortar-brick contact and water distribution measurements during the first few hours post manufacture of the joint. Thermal neutrons are scattered by hydrogen atoms substantially more strongly than any other chemical element usually present in bricks and mortars. Consequently, water in masonry can be accurately detected by neutron radiography. A purpose made brick laying device was used to lay the brick onto the fresh mortar bed within the neutron beam; this enabled water movement from the mortar to the brick to be monitored during the first few minutes of contact. Water changes were measured at two locations within the bed joint; in the middle of the mortar joint and 2mm below the surface of the lower brick interface, within the body of the brick. The choice of lower brick is interesting since traditional attempts of measuring water movement from the mortar to the brick have been found generally by placing a brick on a free water surface or on a wet mortar bed and monitoring water flow to the upper unit. The difference in bond formation between the lower and the upper unit and the possible mechanisms which influence bond formation have been discussed in Chapter 4. Groot reports that the influence of gravity on water transport is negligible, since he observed high water contents in bricks both below and above the joint.

Groot's experimental design included three types of unit with very different water absorption characteristics; fine porous extruded clay brick, coarse porous moulded clay brick and a calcium silicate unit. These were bonded with two types of mortar, Portland cement:sand 1:4½ air entrained mortar and masonry cement:sand 1:3 air entrained mortar.

Groot observed that the time period during which water was extracted from the mortar, was vastly different for each type of unit, irrespective of mortar type; a stable water content in the mortar was attained for the extruded brick after 1-hour, for the moulded brick after 5-minutes and the calcium silicate brick after approximately 4-hours. He determined that the brick with highest measurement of initial suction rate (IRA) exerted suction for longest period, the moulded brick with lowest IRA ceased to extract water in shortest period.

Groot found that the final water content in the core of the mortar is comparable for the extruded and moulded brick types for a given mortar type (10% and 14% respectively). Groot suggests that the hardening conditions of the mortar in the core of the joint are the same for each brick type. The suction characteristics of the calcium silicate unit is markedly different, the final water content in the core of the mortar is less than 5%. For all units, the effect of water retention from the masonry cement mortar was shown to be significant.

Having demonstrated that water abstraction from the mortar could be measured using neutron radiography, Groot further attempted to calculate the water flow velocity in the mortar. He calculated initial water velocities upon contact for the moulded brick between 0.25–0.3 mm/s, 0.05-0.12mm/s for the calcium silicate and 0.05-0.08 for the extruded brick. He observed that water velocities diminish steeply with time, which suggests that if velocity of flow is considered to be a contributory factor towards bond strength development, that the initial seconds of contact are critical.

From his experiments, Groot was able to plot the mass of water absorption with time, calculated from the measured decrease in water cement of the mortars. The resultant graphs produced water absorption profiles very similar to those demonstrated in Chapter 7. The graphs show a steep initial slope from the origin, characteristic of high water flow velocity, which then become asymptotic with time. The moulded brick depicts a much steeper gradient than both the calcium silicate and extruded brick. Groot argues that with high water velocities, more fine material will be transported to the interface than with low velocity of flow.

From similar tests, Groot reported that the process of suction rate adjustment has little influence on the process of absorption, the only difference being the final water content in the mortar. The suction of docked bricks is apparently high enough to absorb nearly the same volume of water out of the cement mortar as the dry brick

Generally, Groot found that masonry cement mortars perform less well in bond strength than the Portland cement mortar. The extruded and calcium silicate units perform better with masonry cement mortar than with Portland cement mortar. Groot attributes this to the decrease in water in the masonry mortar, which reduces water at the interface and therefore increases the water cement ratio. Groot maintains that unfavourable mix composition at the interface for moulded brick is caused by transfer of fine cement and ground limestone particles by brick suction. Groot also observed that the calcium silicate units performed poorly in bond strength with the Portland cement mortar, due he suggests to the large water content in the surface of the unit. However Groot reports that final bond strength values were adjusted, to make allowance for the reduced cement content of the masonry mortar. This approach seems fundamentally flawed, since it assumes that there exists a direct relationship between cement content of the mortar and bond strength. As discussed in Chapter 2, any such observed relationship is considered to be a function of the particular method used in the application of bond strength testing.

The low bond strength of masonry cement mortars compared with Portland cement:lime mortars was also reported by Matthys[124]. He found that regardless of both mortar designation and unit type, the Portland cement mortars exhibited significantly higher bond strength than their code equivalent masonry cement mortars. Matthys also observed that the coefficient of variation of bond strength results measured by bond wrench for masonry cement specimens was also significantly higher than Portland cement:lime mortars.

Examining the results of Groot's experiments, it can be observed that there is a clear difference in water transport behaviour between different types of clay and calcium silicate bricks. The moulded brick sample shows more water at the interface than the body of the mortar while the opposite is true for the extruded unit. For the moulded unit, the quantity of water at the interface appears more steady state, while for the extruded unit, there is a rapid initial rise in water content at the interface. The moulded unit depicts a high initial loss of water at the centre of the mortar bed. Both the extruded and moulded clay bricks show that water is quickly dissipated within the body of the brick, while the calcium silicate unit shows long lasting concentration of water in the vicinity of the interface. This suggests that high initial water velocities may not necessarily lead to high water abstraction by the brick if the brick does not have the capacity in the boundary layer to accept this water.

In an earlier paper, Groot[125] described work that examined the water distribution over the cross-section of the mortar joint with respect to the hardening conditions at the interface.

Groot concurs with the findings of this study that the measurement of IRA represents only one point on a true water absorption curve and as such, does not characterise the true mass-time relationship of water absorption. Groot also concluded that there was no direct relationship between IRA and bond strength, as discussed in Chapter 6.

Groot makes reference to a device which enables the continuous water uptake to be measured, but does not expand on the workings of such apparatus. Attempts to measure the continuous and uninterrupted water absorption of a unit from a free water surface were shown in Chapter 7 to be fraught with complexity. Groot's idealised apparatus suspends the unit's bed-face in a reservoir of water having a constant level, and measures water absorbed by the unit by means of a load cell which the unit is suspended from. The reality of such a measurement is that as the unit approaches the free water surface there will be the inevitable surface tension jump of the water towards the unit surface, which will be measured by the load cell device. The extent of surface wetting of the unit, as opposed to water absorption into the pores of the unit, will further complicate readings around the origin of the water absorption verses time graph. Furthermore, there will be an element of up-thrust experienced by the brick. It is also impossible to differentiate between the quantity of water that has been absorbed or partially absorbed into the surface pores of the brick and that proportion of water remaining in the reservoir.

In a further study Groot and Larbi[126] modelled capillary water pressure and water transport for cylindrical capillaries (bricks) and water-containing particle systems (mortars) in an attempt to explain the complex process of water transport of mortar mix water by brick background suction across the bonding interface. They were able to show that coarse pores exert low capillary pressure and fine pores high capillary pressure, as shown by the following equation for water pressure:

$$P = 2\,Q/r$$

Where **Q** = surface tension N/m and **r** = radius of capillary in m

Water transport in an open uniform capillary from a free water surface is governed by capillary force, water to tube friction and gravity. The mass of water absorbed after time t is given by the following equation:

$$M = c.r^{2.5}\sqrt{t}.p$$

Where **M** = mass in kg, **r** = radius (m), **t** = time (sec), **p** = density ($kg/m^3$) and **c** is the constant pi $\sqrt{}$ ( Q/2n), where n is the dynamic viscosity ($Ns/m^2$).

As can be demonstrated, the effect of pore diameter on mass of water transferred is considerable; a factor of 10 in pore radius corresponds to factor 316 for mass water transported.

The authors acknowledge that modelling brick pore structure as a system of open capillary tubes is of course an idealisation; the pore structure is far more complex dependent on pore shape, varying pore diameter, pore orientation, interconnections of capillaries and closed pores. As such, they argue that bricks with equal IRA values may reflect pore systems with different pore size distributions and pore volumes. This again concurs with the work described in Chapter 7.

The authors recognise that while brick capillary pressure exerts suction forces on the mortar, the process of decreasing water content of the mortar and the possible movement of particles towards each other causing compaction and densification of the mortar, considerably influence the capillary pressure of the water in the mortar.

Groot and Larbi study the theory of capillary pressure due to liquid cups which form due to surface tension of the water film between two or more particles a small distance apart. The theory shows that capillary water pressure increases as the distance between particles reduces as the mortar is compacted. As the mortar loses water, the capillary pressure will increase. The model also shows that for decreasing particle diameter, capillary water pressure in the mortar will increase; in the case of different particle dimensions, the particle with the smallest diameter will determine the capillary pressure. For a mortar, which requires considerable mix water content to achieve desired workability, the water to fines ratio (cement and additions such as hydrated lime, ground limestone) may range between 0.8 and 1.3. The authors maintain that the water loss from a fresh mortar due to brick suction may range from between 10 and 80%. Given the water pressure theory described above, the water pressures generated due to compaction and densification may be significant. From the specific surface value of the mortar fines and the initial water-fines ratio, the distance between the particles can be estimated through the determination of water film thickness around the particles. Therefore if a masonry cement or lime cement mortar is used, the average thickness of the water film will be half that of a Portland cement mortar since the specific surface of these binders is about twice as high of that of the Portland cement. It should be noted that the high specific surface of masonry cement mortar is generated by high quantities of ground limestone. The specific surface of lime used in lime:cement mortars is higher than ground limestone.

Groot and Larbi recognise that during a period of high initial flow rate, fine material may move with the flow from the mortar to the brick. As a result of the initial water loss, the particles will move to each other and the cup angles will decrease, leading to high water pressures in the mortar. At this point, the finer particles become immobilised in the dense packing of the mortar. Since masonry cement and lime-cement mortars show both lower initial distances between the particles and finer particles, the initial flow rate and water loss will be lower than for Portland cement mortars.

The authors observed that water-cement ratio decreased gradually with increasing IRA and concluded that water loss gradient for cement mortars is steeper than for the masonry cement mortars.

They state that the bond between hydration products in mortars and non-calcareous substrates such as bricks, are the result of physical forces and that chemical forces are negligible. The bond is formed from a mechanical interlock and to a lesser extent van der Waal's forces of attraction.

The outcome of their investigation using both microscopic analysis of the interfaces and capillary pressure theory show that significant difference in mortar brick bond strength for bricks with more or less similar high IRA's may be caused by flow reversal of the water from the brick to the mortar. They suggest that the mechanism of flow reversal could occur after partial hydration of the mortar, if the bricks contain coarse pores filled with water, which is free to be drawn back into the mortar by flow reversal exerted by capillary pressure of the mortar structure. The quantity of water flowing in reverse is higher for fired clay bricks; lime:cement mortars show higher reverse water flow than Portland cement and masonry cement mortars.

The above observations hold significant relevance to the findings of this research programme. Firstly the concept of flow reversal could explain why early bond strength measurements shown in Chapter 4 showed a peak in bond strength post manufacture of the joint, which was coupled with the preferential failure plane becoming predominantly lower plane failure. The phenomena reported in the Literature to Chapter 4 regarding workers finding preferential failure planes could be explained by the flow reversal process, which could be further influenced by the effects of gravity; bricks above the joint release their acquired water back to the bonding interface more readily under the flow of gravity than bricks below the joint.

### 9.1.2 Microstructure of bonding interface

Sugo, Page and Lawrence[127] examined the de-bonded surface of the mortar bed using both optical and scanning electron microscopy techniques (SEM). They examined masonry cement mortars in combination with clay units having a low absorption rate defined by the IRA test. The authors compared results with a previous study using Portland cement lime mortars and observed no significant difference between residue micro-constituents formed at the interface between the two different mortars.

The authors recognise (but do not reference) numerous reports in the literature reporting poor bond strength performance from masonry cement mortars. They attribute this to high levels of entrained air and low Portland cement content. Groot[1] using X-ray diffraction found that ground limestone constituent of the masonry cement mortar was preferentially transferred to the bonding interface by brick background suction. Hime and Martineck[128] reported numerous case studies observing poor bond strength performance and water leakage from masonry cement mortars. They attributed this to poor interface contact, low paste volume and low levels of hydration resulting from the reduced quantity of mix water necessary to obtain workable mix. This was due to high levels of entrained air due to the presence of air entraining agents in the masonry cements. They also noted the difference in plasticity between ground limestone and hydrated lime and differences in the surface saturated dryness (SSD) values, these being 25% and 55% of dry mass for ground limestone and hydrated lime respectively.

Sugo et al. made an estimate of the mortar-bed joint water cement ratio, by separating the couplets 1-hour after manufacture and removing the mortar. The moisture retained by the mortar was attained by drying to a constant mass at 60°C. The 1-hour time period was selected as a reasonable period to allow for brick suction effects to reach a minimum.

Examination of the masonry cement mortar using the SEM technique identified the main constituents of the mortar to be calcium silicate, limestone and gypsum.

The workers also found during bond testing that the nature of bond failure occurred predominantly in the top plane. This is contrary to the majority of results recorded for this research programme which reported predominantly lower plane failure. It was discussed in Chapter 5 that the preferential failure plane could be dependent upon the particular units used in experimentation, since levels of water uptake by the unit and subsequent release of water to flow reversal by both gravity and mortar hydraulic suction will all influence the strength of the bond at the interface.

Inspection of the corresponding mortar side of the specimens revealed a very thin layer of paste which had migrated to the interface; they found that there was a region of voids between the interface layer and the aggregate matrix behind; the large number of spherical voids were reported to be indicative of air entrainment. Examination of the mortar surface following bond rupture revealed a dense, smooth microstructure reflecting the topography of the brick face. The micro-constituents appear to be rod-shaped calcium silicate hydrate (CSH) and $Ca(OH)_2$ crystals and some ettringite needles The rod shaped CSH needles appeared preferentially aligned growing from the central point and spreading out towards the surface of the unit. The rod shaped morphology of the mortar surface was restricted to the first and second layer of cement particles.

Sugo et al. report that the initial water-cement ratio for the masonry cement and the Portland cement are 1.04 and 1.79 respectively. After 1-hour these have reduced to 0.24 and 0.71. Consequently, a significant proportion of the initial mix water in the mortar is transported to the unit. The water cement ratio of 0.24 is lower than the estimated required amount observed by Bye[129] of 0.38 necessary to complete hydration. This agrees approximately with the work reported by Pyle in Chapter 3, which stated that approximately one-third of the mix water was necessary to satisfy cement hydration. The bricks in this experiment were oven dried before use in an attempt to reduce variation in brick absorption characteristics. Consequently, it is considered that the residual moisture content of the bricks was unnaturally low.

The authors observed that a weak zone can be formed at the interface, associated with poor build up of cementitious material. Examination of the microstructure of the mortar at the interface shows a high level of air entrainment resulting in a foam like structure between the aggregate particles.

The micro-constituents were observed to be rod-shaped CSH products, $Ca(OH)_2$ and some ettringite. The rod shaped CSH products were preferentially aligned towards the unit surface and resembled "hair brush" morphology. Immediately behind this layer the morphology of CSH changed to a fluffy type. These micro-constituents were found to be very similar to the authors previous parallel study with cement lime mortars.

No difference in the degree of hydration could be observed despite the lower water-cement ratio. In the body of the mortar remote from the interface, fluffy type CSH products were observed with fewer deposits of $Ca(OH)_2$ crystals compared to the Portland cement:lime mortar. The absence of lime in the masonry cement mortar gives unfavourable conditions for the formation of large crystals due to lower moisture content. The reduced quantity of $Ca(OH)_2$ crystals within the bulk of the mortar may influence the long term strength characteristics since it reduces the capacity of the mortar to undergo autogenius healing and strength gain through carbonation.

The mean bond strength for the masonry cement mortar was approximately a third of the Portland cement mortar. The reduced bond strength of the masonry cement mortar would appear to be primarily due to the presence of air bubbles formed at or near to the bonding interface and also the reduced fluidity of the paste due to the reduced quantity of mix water required. The authors reported that there was no significant build up at the interface of any fine material carried by brick suction forces.

Sugo Page and Lawrence[130] in a later study attempted to correlate bond strength performance to the macro and micro-constituents formed at the bonding interface. The authors were also able to identify several key stages in the formation of bond development. They adopted four unit types; extruded clay brick, dry pressed clay brick, concrete and calcium silicate units. The clay bricks were oven dried before use while the concrete and calcium silicate units were allowed to reach equilibrium water content in the laboratory. These were then combined with three mortar types; 1:6 cement:sand, 1:6 cement:sand with methyl cellulose thickening agent and a 1:1:6 cement:lime:sand mix. This allowed for the comparison of the effects of both lime and the thickening additive against the 1:6 cement:sand mix.

The quantity of moisture absorbed by the units for each brick-mortar combination were monitored by separating the mortar bed after 1-hour and determining the residual moisture content of the mortar and unit. The microstructure of the units were examined using both optical and scanning electron microscopy techniques. The surface texture of the extruded unit was observed to have smooth appearance with few small round openings. The dry pressed clay unit showed a highly crazed surface texture whilst the concrete unit had a fine pore structure associated with cement hydration products. The calcium silicate unit had a relatively coarser mesh structure formed by fibrous nature of the calcium silicate hydrate (CSH), produced by the autoclaving process. The paper shows interesting photomicrographs of the different unit bed faces.

The authors found that the 1:1:6 cement:lime:sand mortars retained the most moisture and developed the greater tensile bond strengths. The dry pressed and calcium silicate units showed that a distinct layer of cementitious material had been transported to the interface. Couplets formed using these types of unit tended to fail within the body of the joints as opposed to the extruded and concrete units, where failure was seen to occur at the interface.

The authors draw the following conclusions from their findings: the strength and mode of failure are influenced by the volume of paste (the non-lime mortars lacked volume), the transport of mortar fluids and cementitious material to the interface and the density and degree of hydration of the micro-constituents.

The authors further postulate the processes which occur during bond formation. The initial contact between the brick and the wet mortar bed initially wets the brick surface; wetting of the surface must occur before brick capillary suction can be initiated. Wetting of the brick surface is also necessary before growth of $Ca(OH)_2$ or CSH can take place on the brick substrate. At a molecular level, the brick surface is covered with a layer of adsorbed atmospheric gases. The path taken by this adsorbed gas may influence bond strength as the laying process and water absorption of the brick squeeze out this air. The authors demonstrated with polished sections of mortar that there were regions of entrapped air located adjacent to, but behind the interface layer. They suggested that there is a mechanism whereby paste is transported to the interface around the voids.

Capillary suction of mortar fluids by unit suction takes place immediately upon contact between mortar and brick. The transfer of water into the brick lowers the suction potential of the brick and increases the corresponding suction potential of the mortar. The process continues until equilibrium in suction potentials is reached; this process is highly localised and will induce a sharp moisture gradient in the interface zones. Capillary suction can potentially transport solids to the interface, which will induce plastic shrinkage. The authors found that the amount of water removed from the mortar was not dependent upon Initial Rate of Absorption of the unit. The extruded clay, concrete and calcium silicate units had similar IRA values yet produced very wide range of 1-hour moisture contents. This finding further confirms the conclusions drawn in Chapters 6 and 7 that the IRA test is not an accurate indicator of a units water abstraction potential.

Loss of moisture to the environment during the initial brick-mortar contact period will influence the cement hydration that occurs over several days. As discussed in Chapter 2, cement hydration no longer takes place when the relative humidity reduces below 80%.

The authors maintain that given the right conditions, transport of fluid within the mortar will create a build up of mortar fines along the brick-mortar interface. The critical properties of the unit and mortar which influence the amount of solid transfer are rate and volume of capillary flow, the particle size of the fines and rheology of the mortar paste.

The transport of solids to the interface provides continuity of contact between the two materials. This build up of fine material at the interface is likely to reach an optimum; insufficient amounts will lead to adhesive failures, while excessive build up will lead to lowering of the cohesive tensile strength of the mortar layer adjacent to the interface (paste depleted layer). This phenomena was observed with the dry pressed and calcium silicate bricks. The build up of fines at the interface also forms a barrier to water flow reducing both fluid and solid transfer.

Volume changes within the paste take place due to de-watering of the mortar by brick suction. Once the mortar becomes unsaturated, the coarser aggregate particles form a three-dimensional grid, which will resist further volume change. Shrinkage of the paste surrounding the aggregate will take place up to a point where the moisture content is such that the small particles can no longer re-arrange themselves. The point of optimum packing of the cement particles marks the end of plastic shrinkage. If moisture extraction occurs while the paste remains sufficiently plastic to accommodate volume change then capillary suction and shrinkage are beneficial to bond strength development since the cement particle spacing is decreased. If suction is too high, then there will be insufficient water available for full cement hydration. Conversely, if capillary suction is too low, relative to the mortar water retention capacity, then a high water-cement ratio will result, leaving a poorly developed contact layer between the unit and the mortar.

Following plastic shrinkage, autogenous shrinkage due to cement hydration will take place followed by drying shrinkage due to partial dehydration of the calcium silicate hydrate structure as moisture is lost to the environment. Furthermore, carbonation of lime will also contribute to shrinkage of the mortar matrix. The authors concur with the conclusions of this study, that combined shrinkage effects may cause a reduction in bond strength and mortar cohesive strengths with time.

Formation of CSH products due to cement hydration, will increase the suction potential of the mortar since CSH products have high surface area and are hygroscopic in nature. This produces flow reversal effects, as noted by Groot. Flow reversal of water away from the unit back the mortar possibly has a positive influence on bond strength by providing moisture to aid the hydration process. Another potential influence of moisture flow reversal is to change the chemistry of the reversal solution. Mix water absorbed by the masonry unit may dissolve adsorbed $CO_2$ and $SO_3$ compounds. These ions may further influence the micro-constituents formed at the interface. A solution rich in carbonate ions may lead to formation of calcium carbonate build-up at the bonding interface, which will create a denser mortar matrix. Sulphate ions may also lead to the formation of ettringite, although the authors consider that the high volume change associated with crystallisation may actually hinder the bond strength development. Carbonation reactions take place due to ingress of dissolved carbonic ions or atmospheric $CO_2$. Carbonation $Ca(OH)_2$ causes a cementing reaction which densifies the brick-mortar interface and provides a mechanism for crack healing.

Furthermore, the authors argue that although rapid hydration of tri-calcium aluminate, forming ettringite, occurs immediately upon contact between Portland cement and mix water, the hydration of tri-calcium silicates $C_3S$ and di-calcium silicate $C_2S$ does not occur until the end of the dormant period, which they report as taking place approximately 4-6 hours after joint formation. They maintain that the rapid increase in the rate of CSH formation at the end of the dormant period is the beginning of the bond formation. This was also reported by Reda and Shrive [131] who argue that while the role of ettringite formation is clearly significant in the development of bond strength, calcium silicate hydrate formation should lead to even stronger bond strengths since CSH is inherently stronger than ettringite. Hydration of the $C_3S$ and $C_2S$ components will continue until they are consumed or until hydration ceases due to insufficient moisture availability. The above findings further support the findings reported in Chapter 4, which demonstrated that there was a significant increase in bond strength around 6-hours post manufacture of couplet joints.

Reda and Shrive [131] in their study to examine the effect of using fly ash in masonry mortars upon bond strength observed ettringite crystals in the masonry pores under SEM and argue the important role ettringite plays in the formation of bond strength. They described the ettringite crystals as hexagonal, needle-like shape with a diameter of about 0.05 μm. They observed more ettringite formation in the body of the mortar for moist cured samples while no difference in the amount of crystallisation could be detected for the dry cured samples. Unreacted gypsum crystals were observed beside ettringite crystals in the moist cured samples, whereas no gypsum was observed for the dry cured samples. They suggest that this is due to lack of water to allow for full hydration of the gypsum.

Sugo et al. conclude that unit suction of the mortar fluids and associated transport of solids to the brick-mortar interface form an important role in the development of the bond. The mortar de-watering effects the adhesive and cohesive strength and mode of failure of the joint by the development of a uniform contact layer and by a reduction in the high initial water-cement ratio of the mix.

The authors argue that the interaction between the unit and mortar limit the usefulness of the IRA test; a more complex model incorporating unit suction, rheology of the paste and its suction properties are required.

Lawrence and Cao[132] in a further paper examined the interface between a specific type of clay brick with three different initial moisture contents and four cement pastes, including plain cement, limed cement and paste with air entraining agents, by means of scanning electron microscopy and associated techniques including energy dispersive X-ray analysis (EDXA) and energy wavelength dispersive spectrometry (EWDS). The authors attempted to relate the interface microstructure of each array of samples with the bond strength measured by bond wrench tests.

The effect of lime on the interface microstructure was to facilitate the formation of the initial calcium rich film and to increase the amount of calcium hydroxide at the interface. The disadvantage of limed cement mortar was to lower strength at early ages due to the higher percentage of calcium hydroxide at the interface.

The microstructure of the saturated brick interface showed coarser hydrates than the dried brick, indicating high porosity. The authors were able to show that brick initial moisture content had a marked affect on bond strength. The lowest measured bond strength occurred with units having 14% moisture content while maximum bond strength occurred for units having 6% moisture content. They found that bond strength dropped sharply when moisture content exceeded 11%. Lawrence and Cao state that the optimum bond strength is reached for bricks having an initial moisture content of between 6% and 8%. The presence of lime was found to lower the bond strength at early age (7 Days).

Their study demonstrated that the initial moisture content of the brick affects the bonding due to a change in the units suction characteristic. For most types of mortar, the water cement ratio is around unity. At this high value, porosity is high and low bond strength would be expected. When the brick is saturated, its suction is low and there is increased porosity due to the high water-cement ratio.

## 9.2 Concluding Remarks

The literature provides an insight into the actual processes which occur within the mortar joint during the period of bond strength development. It has always been maintained by researchers that the transfer of water from the mortar to the brick by brick suction effects, results in fine cementitious constituents being preferentially amassed at the bonding interface. It is the hydrated crystalline structure that these constituents develop which have been attributed to the formation of the bond, by generating a mechanical interlock between the mortar and the pores in the boundary layer of the masonry unit.

However, microscopic investigations described above suggest that this layer of constituent material is only one or two particles thick, behind which lies a weaker layer consisting substantially of voids. While the transport of solids to the interface does provide continuity of contact between the body of the mortar and the brick, it remains hard to accept that the magnitude of bond strength can be attributed to the strength of such a finite layer.

It is possible that observations of build up of constituent material at the interface are due to fines being carried in solution in the mix water, as opposed to fines being bodily drawn through the mortar by brick suction forces. Furthermore, some authors[123] report significant build-up of fine material at the interface while others[127] report no significant deposition of fine material.

The concept of reverse capillary suction exerted by the mortar and reversal of flow from the brick to the mortar is of most significance, since it indicates an even more complex mechanism than had previously been considered. Traditionally, researchers have examined water flow in one direction, from the mortar to the brick. If it can be shown that water absorbed by the unit is able to return to the bonding interface to aid in the hydration process, this would help to explain the phenomena of preferential failure planes. Similarly, excessive saturation of the constituents at the interface will result in high water-cement ratios and a loss in cohesive strength of the mortar, as has been shown for calcium silicate units.

Given the evidence, it would appear that there is a short window of opportunity in which optimum bond strength can be initiated and this is supported by Groot's work, which demonstrated that the rate at which water is extracted is significant. As water extraction takes place, loss of water in the mortar together with the movement of particles towards each other, creates a rapid reverse capillary suction force within the mortar. Densification due to packing, which may be more intense at the bonding interface due to the presence of fine material, prevents further movement of water. This point of optimum packing marks the end of plastic shrinkage. If the excess mix water, not required in the cement hydration, has been evacuated from the mortar matrix before this congealing takes place, then the cementitious particles will have closer spacing and optimum inherent cohesive strength. Any remaining chemically unbound water, enclosed in the mortar matrix will, in time lead to drying shrinkage.

What happens to the mix water once it has been exiled from the mortar is dependent upon the pore structure and diffusivity of the particular unit. It is considered that a residual quantity of water retained at the interface will help hydration and aid curing.

The formation of the crystalline structure, to which bond strength has traditionally been attributed, cannot fully explain the rapid initial development of the bond within the first few minutes of contact between brick and mortar. It is considered here that the mechanism of plastic shrinkage must induce a lateral compressive stress between the mortar and the brick surface, contributing to the early bond formation. This action becomes more permanent once the mortar begins to stiffen. The ability of the mortar to resist both tensile stress applied normal to the bed joint and stress induced by drying shrinkage then becomes a function of the mortars cohesive strength to which calcium silicate hydrate and ettringite play an important role.

## References

122 Groot, C. Effects of water on mortar-brick bond. Delft University of Technology, Netherlands. 1993.

123 Groot, C.J.W.P. First Minutes Water Transport from Mortar to Brick. Proceedings of the 9th International Brick/ Block Masonry Conference. Berlin, Oct.1991.

124 Matthys, J.H. Flexural Bond Strengths of Portland Cement Lime and Masonry Cement Mortars. Proceedings of the 8th International Brick/ Block Masonry Conference. Dublin 1988

125 Groot, C.J.W.P. Aspects of Mortar-Brick Bond. Proceedings of the 8th International Brick/ Block Masonry Conference. Dublin 1988

126 Groot, C.J.W.P. and Larbi, J.A. The Influence of Water Flow (Reversal) on Bond Strength Development in Young Masonry. Proceedings of the 12th International Brick/ Block Masonry Conference. Madrid 2000

127 Sugo, H.O., Page, A.W. and Lawrence, S.J. The Development of Mortar/ Unit Bond. 9th Canadian Masonry Symposium. 2001.

128 Hime, W.G. and Martineck, R.A. Masonry Cement – Peas in a pod, or apples and turnips?. Proceedings of the 7th North American Masonry Concference, India. 1996

129 Bye, G.C. Portland cement, composition, production and properties. Pergamon Press, Oxford, 1983.

130 Sugo, H.O., Page, A.W. and Lawrence, S.J. A study of Bond and Mortar Microstructure Developed Using Masonry Cement. Proceedings of the 12th International Brick/ Block Masonry Conference. Madrid 2000

131 Reda, M.M. and Shrive, N.G. Enhancing masonry bond using fly ash. Masonry International, vol 14, No.1, 2000.

132 Lawrence, S.J. and Cao, H.T. Microstructure of the Interface Between Brick and Mortar. Proceedings of the 8th International Brick/ Block Masonry Conference. Dublin 1988